ASCE Manuals and Reports of Engineering Practice No. 77
WEF Manual of Practice FD-20

Design and Construction of Urban Stormwater Management Systems

ADMINISTRATIVE RECORD INDEX -
DOCUMENTS - STORM WATER TECH.
FOLDER: 4, ITEM # 1

WATER ENVIRONMENT FEDERATION
AMERICAN SOCIETY OF CIVIL ENGINEERS

R0020813

STATE OF CALIFORNIA
CALIFORNIA REGIONAL WATER QUALITY CONTROL BOAR
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ASCE Manuals and Reports of Engineering Practice No. 77
WEF Manual of Practice FD-20

Design and Construction of Urban Stormwater Management Systems

Prepared by
The Urban Water Resources Research Council of the American Society of Civil Engineers and the Water Environment Federation (formerly the Water Pollution Control Federation)

Published by the
American Society of Civil Engineers
345 East 47th Street
New York, New York 10017-2398
and the
Water Environment Federation
601 Wythe Street
Alexandria, VA 22314-1994

R0020815

ABSTRACT

Design and Construction of Urban Stormwater Management Systems (Manual of Practice No. 77) updates relevant portions of the ASCE/WPCF Manual of Practice No. 37, *Design and Construction of Sanitary and Storm Sewers.* This update is necessary due to the many changes taking place in the field such as the use of microcomputers and the need to control the quality of runoff as well as the quantity. In order to broaden the base of experience reflected in the Manual, each chapter was prepared by one or more authors with experience and expertise in the particular subject area. Thus, the Manual aids the practicing engineer by presenting a brief summary of currently accepted procedures relating to the following areas: 1) Financial services; 2) regulations; 3) surveys and investigations; 4) design concepts and master planning; 5) hydrology and water quality; 6) storm draining hydraulics; and 7) computer modeling.

Library of Congress Cataloging-in-Publication Data

Design and construction of urban stormwater management systems/prepared by the Urban Water Resources Research Council of the American Society of Civil Engineers and the Water Enviroment Federation (formerly the Water Pollution Control Federation).

p. cm.—(ASCE manuals and reports engineering practice: no. 77) (WEF manual of practice: FD-20)

Includes bibliographical references and index.

ISBN 0-87262-855-8

ISBN 1-881369-21-8

1. Urban runoff—Management. 2. Storm sewers—Design and construction. I. American Society of Civil Engineers. Urban Water Resources Research Council. II. Water Environment Federation. III. Series: ASCE manuals and reports of engineering practice: no. 77. IV. Series: WEF manual of practice: no, FD-20.

TD657.D47 1992

628'.21—dc20 92-36519

CIP

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Library of Congress Catalog Card No: 92-36519
ISBN 0-87262-855-8
ISBN 1-881369-21-8
Manufactured in the United States of America.

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R0020820

HISTORY OF THE MANUAL

The Urban Water Resources Research Council (UWRRC) of the American Society of Civil Engineers (ASCE) has long been a leader in the transfer of urban drainage technology to the engineering community. A major part of its efforts has been the publication of more than a dozen books related to the general subject of urban storm drainage, most of which are the proceedings of a series of Engineering Foundation Conferences. Many of these books are used as standard references in the field.

The last Manual of Practice related to urban storm drainage was Manual of Practice 37, "Design and Construction of Sanitary and Storm Sewers," published jointly in 1969 by ASCE and the Water Environment Federation (formerly the Water Pollution Control Federation). In 1982, it was supplanted, for sanitary sewers, by Manual of Practice Number 60, "Gravity Sanitary Sewer Design and Construction," again jointly published by ASCE and WEF.

In an attempt to fill the void created by the publication of Manual of Practice 60 (which did not address storm drainage), the UWRRC established a Task Committee, chaired by Mr. Richard Lanyon, to begin drafting a manual of practice on urban storm drainage. The chairmanship of the Task Committee was subsequently assumed by Mr. Jonathan E. Jones, who has seen the Manual through to its completion.

To broaden the base of experience to be reflected in the Manual, each chapter was prepared by one or more authors with experience and expertise in the particular subject area. The chapters were also extensively reviewed by the authors and other experts in the field, as well as other interested parties, including members of the regulatory and public works communities, and specialists in such diverse disciplines as law, planning, landscape architecture, meteorology, and ecology.

A draft of the complete Manual was prepared and distributed to an ASCE Senior Review Committee and the Technical Practice Committee of the Water Environment Federation. Many valuable suggestions for improvement were received. After revision based on these reviews, portions of the Manual were distributed to both committees for a second review.

The final draft of the Manual was approved for publication by WEF and ASCE in 1991.

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FOREWORD

This Manual of Practice for the Design and Construction of Urban Stormwater Management Systems updates relevant portions of ASCE/WEF Manual of Practice No. 37, "Design and Construction of Sanitary and Storm Sewers," published in 1969. This update was undertaken by the Urban Water Resources Research Council of ASCE for several reasons:

(a) ASCE and WEF published, in 1982, a revised Manual of Practice (No. 60) on "Gravity Sanitary Sewer Design and Construction," which did not address storm drainage.

(b) There have been a number of changes in the field of urban storm drainage, including (1) the virtually universal use of the microcomputer for data organization and analysis, and for the analysis and design of urban storm drainage systems, and (2) the increasing importance of designing urban storm drainage systems for the control of runoff quality as well as quantity.

This Manual is intended to aid the practicing engineer by presenting a brief summary of currently accepted procedures. It is not intended to substitute for engineering experience and judgment, nor is it a replacement for more detailed standard texts and references in the field.

The Manual recognizes that the practice of urban storm drainage is dynamic and rapidly changing, with new techniques, materials, and equipment continuously being introduced, and emphasizes that practitioners in the field must constantly be aware of new developments and modify their practice accordingly. The UWRRC invites comments and recommendations for improvement for possible inclusion in future editions.

The authors recognize that many women professionals are involved in all aspects of the planning, design, and construction of urban stormwater management systems. The use of the masculine gender pronoun throughout the manual has been for the sake of simplicity and brevity, and no other inferences should be drawn.

ACKNOWLEDGEMENTS

The Urban Water Resources Research Council would like first to acknowledge the efforts of Mr. Richard Lanyon of the Metropolitan Sanitary District of Greater Chicago, and Mr. Jonathan E. Jones of Wright Water Engineers, Inc. of Denver, Colorado, without whose leadership, energy and enthusiasm this Manual would never have been completed.

Most of the real work was done by the individual chapter authors, and their work is gratefully acknowledged. Mr. Harry C. Torno worked tirelessly as Technical Editor of the entire book.

Special appreciation is extended to the staff of Wright Water Engineers, Inc. in Denver, Colorado who have worked on virtually every aspect of manuscript preparation.

Finally, the Council would like to thank the following organizations, which generously donated funds to assist with the preparation of the text and figures in the Manual of Practice:

- The Engineering Foundation
- U.S. Department of Transportation/Federal Highway Administration
- U.S. Geological Survey
- U.S. Environmental Protection Agency
- U.S. Soil Conservation Service
- Camp Dresser & McKee Inc.
- Michael Baker Engineering
- CH2M-Hill
- American Concrete Pipe Association

R0020825

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Chapter 1

EVOLUTION OF URBAN STORMWATER MANAGEMENT

I. INTRODUCTION

Urban stormwater management is the conceptualization, planning, design, construction, and maintenance of stormwater control facilities in urban/urbanizing drainage basins, and includes all related political, social, and economic considerations. While this definition does not necessarily involve new construction, it includes such facilities as open channels, curbs and gutters, storm sewers, detention/retention ponds and associated structures, water quality enhancement measures, special structures (energy dissipators, transitions, inlets, etc.) and others. While the distinctions between stormwater management and floodplain planning and management are frequently unclear, they may be defined as:

(a) Stormwater (runoff) management is the planned set of public policies and activities undertaken to regulate runoff under various specified conditions within various portions of the urban drainage system (McPherson 1970). It may establish criteria for control of peak flows or volumes, for runoff detention and retention, or for control of pollution, and may specify criteria for the relative elevations among various elements of the drainage system. Stormwater management is primarily concerned with limiting future flood damages and environmental impacts due to development, whereas flood control aims at reducing the extent of flooding that occurs under current conditions (Walesh 1987).

(b) Floodplain management is the regulation of the nature and location of construction on (or other occupancy of) lands subject to inundation, so that foreseeable (probable) flooding damages will have an average annual value (risk) smaller than some preselected amount.

Federal and state policies and guidance are abundant on floodplain management and flood control, but little Federal and state conceptual design policy guidance exists for local drainage and stormwater man-

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agement. It is essential for the drainage designer to recognize that local policies, principles, and criteria will be the primary forces governing design (even when such criteria are at odds with suggested procedures in this Manual of Practice).

II. NEED FOR THE MANUAL

The American Society of Civil Engineers' (ASCE) most recent Manual of Practice (MOP) pertaining directly to stormwater management is MOP No. 37, "Design and Construction of Sanitary and Storm Sewers" (ASCE 1969), which was published jointly with the Water Pollution Control Federation (as WPCF MOP No. 9). In 1982, ASCE updated MOP No. 37 by publishing "Gravity Sanitary Sewer Design and Construction," ASCE MOP No. 60 (ASCE 1982).

Much of MOP No. 37 is as relevant now as when it was written, and anyone designing storm sewers should be aware of its content, some of which has not been duplicated or updated in this Manual. There was a perceived need to update MOP No. 37, however, because MOP No. 60 does not adequately address urban storm drainage, and because there have been a number of changes in storm drainage practice over the past two decades, including the following:

(a) The proliferation of stormwater retention and detention ponds. A technique that was not widely used thirty years ago is now a common feature of small and large developments and drainage master plans (Malcolm 1982).
(b) Increasing emphasis on the preservation and improvement of natural drainageways. Stream master planning, with an emphasis on "designing with nature," is widely practiced (Rickert and Spieker 1971; ASCE 1978; Sheaffer et al. 1982).
(c) Widespread use of the terms "minor" and "major" (see Glossary) in drainage system planning and design to describe, respectively, the carefully planned, designed and constructed urban storm drainage system; and the overflow path(s) followed by the rarer large flows that exceed the capacity of the minor system.
(d) The use of personal computers and their accompanying hydrologic, hydraulic, and data management software. For a relatively modest expenditure, any engineer can now have a system powerful enough to complete calculations in minutes that would have taken weeks twenty years ago. This has also caused problems, particularly where there has not been adequate computer program (model) calibration and verification (Huber 1986), and where the engineer has blindly relied on computer-generated results without the review normally applied in more conventional methods of design.
(e) The increasing importance of evaluation and mitigation of adverse impacts of urban runoff on receiving water quality. Quality enhancement measures are becoming routine components of stormwater control.
(f) Increased structural complexity of facilities commonly used in drainage systems. Structural and geotechnical engineers are now routinely retained to provide specialized consulting during design.

(g) Changes in construction materials and methods. For example, soil cement, roller compacted concrete, structural plastics and geotextiles, which were rarely used in drainage projects when MOP No. 37 was published, are now common materials in stormwater management projects. Construction sequencing and management have advanced considerably with the advent of personal computers.

Additional reasons for updating the Manual include:

(a) Professional liability concerns increasingly are influencing design practice. Every designer faces the threat of litigation if his design is perceived as less than the state-of-the-art or—practice. This has resulted in increased attention to the preparation of comprehensive, understandable, and practical plans and specifications (Wells 1984).
(b) Useful sources of information for the designer have multiplied, and now include federal, state, and local governments, universities, and commercial sources (for, for instance, rainfall, streamflow, and land use data). Computer-aided drafting and design (CADD) is rapidly changing the design process.
(c) Regulatory involvement has increased, along with the need to protect the environment, encourage public participation in decision making, and involve specialists from many disciplines in the design process (Spirn 1984).

Denver, Colorado—Attractive, multi-purpose wet retention pond with an adjoining golf course and trail. The water quality is good, and the design emphasizes safety. Certain pollutants from the golf course wash into the pond and are immobilized in sediments on the pond bottom.

(d) Engineers are increasingly recognizing the need to adopt risk-based decision making (McCuen 1985), rather than relying on fixed, arbitrary design standards (such as a "design storm"). The evaluation of upstream and downstream risks before complying with a general design standard is not only prudent but may be necessary to limit liability and protect the public welfare.

Despite changes in the practice of urban stormwater management over the past two decades, and the use of many innovative techniques, the design of urban drainage systems remains a procedure bound by tradition. Common sense has frequently been overridden by adherence to arbitrary standards. A major goal of this Manual is to encourage creativity and the application of broadly-based, practical thinking. This is particularly important in view of the ever-rising expectations that urban storm drainage practitioners are facing, such as those related to the U. S. Environmental Protection Agency's stormwater permit program.

III. ORGANIZATION OF THE MANUAL

The chapters are arranged to permit the reader to follow sequentially through the planning and design process. Chapter 2 addresses the very important legal, regulatory, and financial factors that drive the drainage decision-making process. Required surveys and information sources are delineated in Chapter 3. Chapter 4 deals with design concepts (including conceptualization of the major and minor drainage systems) and master planning.

With a conceptual drainage plan in mind, the engineer next estimates flows and makes a preliminary evaluation of likely water quality issues. These subjects are covered in Chapter 5. Routing through conveyances, ponds, spill-ways, culverts, and other hydraulic structures is necessary, and Chapter 6 outlines the relevant hydraulic principles.

Chapter 7 provides brief descriptions of modeling techniques and of the most commonly used models, along with discussion of how to properly interpret computer-generated results, with emphasis on the need for calibration and verification.

Chapter 8 summarizes the conveyance design procedure. Chapter 9 addresses special structures and appurtenances such as channel protection, manholes and junctions, check dams, energy dissipators, and other such structures, emphasizing their proper design and maintenance. Chapter 10 deals with the special problems of combined storm and sanitary sewer systems. A key component of nearly every new drainage system is stormwater detention or retention facilities, including measures to improve runoff quality, and they are covered in Chapter 11. Chapter 12 provides general guidelines for integrating runoff quality controls into drainage planning and design.

Chapter 13 discusses the wide variety of materials available for constructing storm sewers and open channels, as well as the important

subject of maintenance. Chapter 14 reviews structural principles and design criteria. Chapter 15 addresses the preparation of construction contract documents (plans and specifications). Factors that influence construction such as phasing, safety, site preparation, de-watering, environmental regulations, and quality control are discussed in Chapter 16.

The Manual concludes with an appendix containing two case studies that illustrate some of the techniques described.

IV. THE INTENDED AUDIENCE

This Manual is intended primarily for engineers charged with the responsibility of planning and designing stormwater management facilities. It also should be useful to other engineers, as well as architects, attorneys, planners, and landscape architects, environmental scientists, concerned citizens, and others.

This Manual is not a drainage criteria manual and does not provide extensive detail. It is intended to provide the designer with an introduction to procedures for designing urban stormwater management systems. When additional detail is either helpful or required, the text provides appropriate references. Readers are reminded that this manual provides a summary of the state-of-practice, rather than the state-of-the-art, of urban stormwater management and design in the United States. Emphasis has been placed on practical, easy to use methods of identifying problems and solutions, rather than on theoretical or experimental procedures which have not been well established.

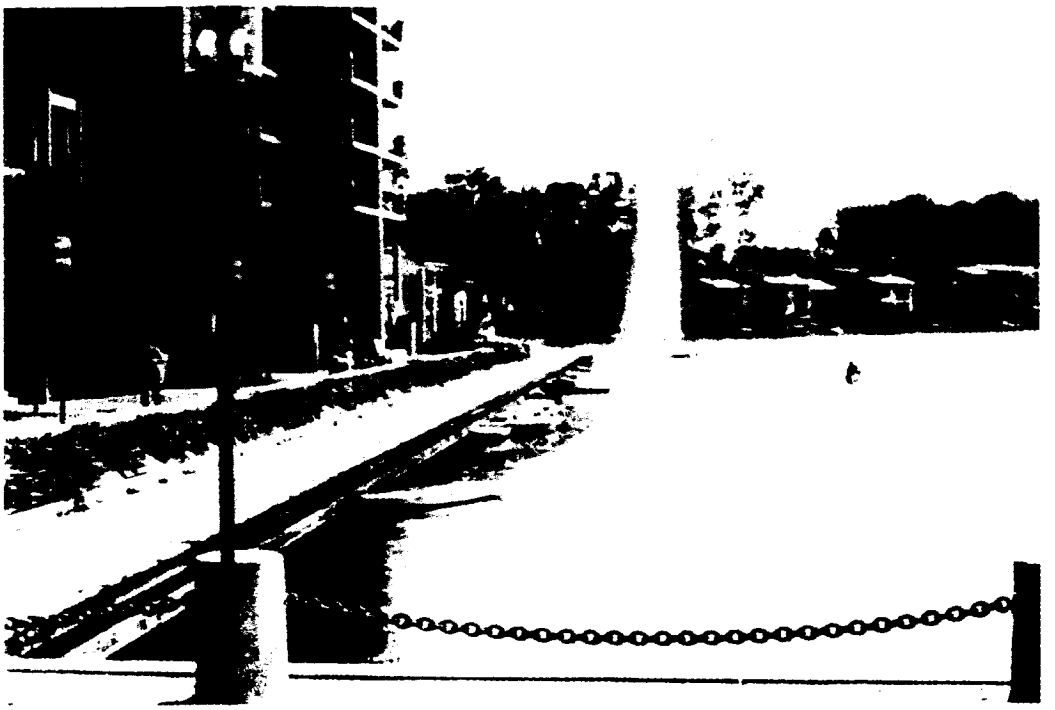
V. THEMES AND PRINCIPLES THAT DRIVE THE DESIGN EFFORT

Certain themes in the manual transcend individual chapters, and may result in some redundancy. These include:

(a) Technically sound drainage projects that are supported by the community evolve through the combined efforts of a large and diverse group of specialists and interest groups, and the engineer should draw on those resources in the planning and design process (Jones and Rossmiller 1990).

(b) Drainage systems are enhanced by facilities that are multi-purpose in nature. Drainageways, ponds, and other facilities can be attractive, provide recreational opportunities, remove pollutants from runoff, provide wildlife habitat, and fulfill other functions, while meeting their primary drainage and flood control objectives (Spirn 1984; APWA 1981; ASCE 1985).

(c) Common sense on the part of the designer is essential. Rigid adherence to particular standards may be undesirable and unwise. Where arbitrary standards are imposed by statute or ordinance, it may be impractical to seek relief, but the engineer should notify the regulatory authorities



Reston, Virginia—Wet pond surrounded by single family and multi-family housing. Note boat dock, fountain, and well-lighted pedestrian access.

if such standards may result in adverse conditions or effects. The engineer must consider the broad picture and evaluate risks on a case-by-case basis.

(d) Despite the widespread availability and adoption of computer models, there is no substitute for experience and judgment. Model calculations must be checked for correctness, and the designer must constantly assess the reasonableness of his results to assure that facilities will function properly (Linsley 1986; Debo and Small 1989; Huber 1990).

(e) Attention to project details is vitally important. An entire chapter is devoted to special structures and appurtenances, and discussions of maintenance, public safety, usability, and long-term performance are provided throughout.

(f) The ASCE Manual of professional practice entitled "Quality in the Constructed Project—A Guideline for Owners, Designers, and Constructors," defines quality as meeting the owner's or user's requirements. Specifically, this publication notes that quality in design and construction may be better understood as:

(1) Meeting the requirements of the owner as to functional adequacy, completion on time and within budget, life cycle costs and operation and maintenance.

(2) Meeting the requirements of the design professional as to the provision of a well-defined scope of work, an adequate budget, and timely decisions by both the owner and the design professional.

(3) Meeting the requirements of the constructor as to provision of contract plans, specifications, and other documents prepared in

sufficient detail to permit the constructor to prepare priced proposals or competitive bids; timely decisions by the owner and design professional on authorization and processing of change orders; fair and timely interpretation of contract requirements from field design and inspection staff; and a contract for performance of work on a reasonable schedule that permits a reasonable profit.

(4) Meeting the requirements of regulatory agencies (the public) regarding safety and health, protection of public property including utilities, environmental considerations, and conformance with applicable laws, regulations, codes, and policies.

(g) Whether or not computer models are used, designers must constantly assess the reasonableness of such factors as:
(1) Frequency of inundation of properties.
(2) Design depths and velocities.
(3) Computed peak discharges.
(4) Stormwater detention volume requirements.
(5) Unit costs of storm sewers and open channels.
(6) Estimated pollutant removal percentages.
(7) Project benefits compared to project costs.
(8) Maintainability.

(h) Many laws and regulations still promote piecemeal, crisis-solving approaches aimed at managing resources within political constraints, or that solve a specific problem without adequate attention to related aspects. Authorities regulating stormwater and other environmental impacts frequently overlap, and their policies are not always consistent. Therefore local stormwater problems often are addressed without evaluating the potential for adverse impacts on upstream or downstream areas. An example would be construction of a relief sewer in an upstream area without evaluating its downstream impacts. Another example would be the uncoordinated development of stormwater management facilities by local land developers, each of whom is responsible for maintaining peak runoff and pollution loads from the site at predevelopment levels. The problems with the piecemeal approach are well recognized (Livingston et al. 1988), and include:
(1) It results in only partial resolution of major flooding problems.
(2) It creates new downstream flooding problems.
(3) Improperly located detention basins may actually increase peak flows rather than reduce them.
(4) Costs of uncoordinated remedial solutions are likely to be much greater than the overall costs incurred had an adequate program been implemented in the first place.

This Manual encourages comprehensive watershed planning, also referred to as "master drainage planning." This approach identifies the most appropriate control measures and optimum locations for the control of watershed-wide runoff and pollution impacts (Schueler 1987). The designer must recognize that individual site development plans or retrofits of drainage facilities in urban areas may be only a part of a larger system and that he must integrate his design into that larger system, particularly with respect to creating upstream or downstream hazards (Urbonas and Glidden 1982; Roesner and Jones 1990). Com-

prehensive watershed planning may indicate that a single stormwater detention facility to control runoff from several land development projects is preferable to on-site detention, and that nonstructural measures such as parkland acquisition, infiltration measures such as porous pavement or swales, and flood-proofing to supplement structural control measures may be preferable to more traditional approaches (APWA 1981). Comprehensive watershed planning offers such advantages as (Urban Drainage and Flood Control District 1984):

(a) Reductions in capital and O&M costs.
(b) A basis for setting priorities for resource allocation.
(c) Reduction in downstream flooding and erosion, particularly in multi-jurisdictional watersheds.
(d) Capability to consider nonstructural measures.
(e) Increased opportunities for recreational and other multi-purpose runoff controls.
(f) Contributions to local land use planning.
(g) Enhanced reuse of stormwater.
(h) Maximum justifiable land occupancies.

It should also be noted that watershed planning can have profound land-use implications. Consider the following examples:

(a) In a newly developing area, the decision-makers who are conducting the master plan want to maximize undeveloped land in the vicinity of major drainageways. One way to do this would be to establish or improve a comparatively small channel that has capacity for, say, the 5-year runoff event. Runoff from larger events would leave the channel and pass downstream as overland flow. Development might then be prohibited within the overland flow zone (floodplain).
(b) The regulation of wetlands under Section 404 of the Clean Water Act can be a very effective land management tool (see Chapter 2).

A major disadvantage of watershed planning is that local governments must perform advanced planning studies to locate and develop preliminary designs for regional stormwater management facilities, and that local governments must finance, design and construct regional stormwater facilities before the majority of future urban development occurs, with reimbursement by developers over build out periods which may range from five to much more than twenty years.

VI. GLOSSARY

The following definitions apply throughout the Manual (although they may not be defined in precisely this manner in other texts).
Storm drainage systems or urban drainage systems—The physical facilities that collect, store, convey, and treat runoff in urban areas. These

facilities normally include detention and retention facilities, streets, storm sewers, inlets, open channels, and special structures such as inlets, manholes, and energy dissipators.

Urban area—Land associated with, or part of, a defined city or town. This Manual generally applies to urban or urbanizing, rather than rural, areas.

Floodplain planning/floodplain management—Technical and non-technical studies, policies, management strategies, statutes and ordinances that collectively manage flood plains along rivers, streams, major drainageways, outfalls, or other conveyances. The federal government normally plays a major role in floodplain planning and management, whereas in urban stormwater management and design, local governments dominate the decision-making process.

Major drainageway—A readily recognizable natural or improved channel that conveys runoff that exceeds the capacity of the minor drainage system, including emergency overflow facilities.

Outfall facility—Any channel, storm sewer, or other conveyance receiving water into which a storm drain or storm drainage system discharges.

Major system—The portion of the total drainage system that collects, stores, and conveys runoff that exceeds the capacity of the minor system. The major system is usually less controlled than the minor system, and will function regardless of whether or not it has been deliberately designed and/or protected against encroachment, including when the minor system is blocked or otherwise inoperable. It may be collinear with, or separate from, the minor system. It should be noted that there are those who object to the use of the terms "major" and "minor" to describe portions of the drainage system, perhaps because these terms imply that the minor system is less important. Other terms (primary system, convenience or basic system, overflow system, major/primary drainage ways, subordinate system, etc.), have been suggested. Major/minor are used in this Manual because they seem to be the most widely used terms.

Minor or primary system—The portion of the total drainage system that collects, stores and conveys frequently-occurring runoff, and provides relief from nuisance and inconvenience. This system has traditionally been carefully planned and constructed, and normally represents the major portion of the urban drainage infrastructure investment. The degree of inconvenience the public is willing to accept, balanced against the price it is willing to pay, typically establishes the discharge capacity or design recurrence frequency of a minor system. Minor systems include roof gutters and on-site drainage swales, curbed or side-swaled streets, stormwater inlets, underground storm sewers, open channels and street culverts. The minor system is considered to end at the point where there are no adverse backwater effects from downstream conditions during discharges smaller than the minor system

design flow (note that, in general, this Manual makes no recommendations regarding the return frequencies that may be used in designing either the major or minor systems, and acknowledges that there has been no satisfactory rationale (in an engineering sense) advanced for the selection of any return frequency for any component of the urban drainage system).

Storm sewer (or storm drain)—Usually, buried pipe that conveys storm drainage. It may include open channel elements and culverts, particularly when drainage areas are large.

Special structures—Those components of urban drainage systems that can be thought of as "features" or "appurtenances" such as manholes, inlets, energy dissipators, transitions, channel slope protection, detention ponds and dams, and outlet works.

Stormwater detention—The temporary storage of stormwater runoff in ponds, parking lots, depressed grassy areas, rooftops, buried underground tanks, etc., for future release. Used to delay and attenuate flow.

Stormwater retention—Storage designed to eliminate subsequent surface discharge. Wet ponds are the most common type of retention storage (though wet ponds may also be used for detention storage).

Master drainage plan—The plan that an engineer/designer formulates to manage urban stormwater runoff for a particular project or drainage area. It typically addresses such subjects as characterization of site development, grading plan, peak rates of runoff, and volumes for various return frequencies, locations, criteria and sizes of detention ponds and conveyances, measures to enhance runoff quality, salient regulations and how the plan addresses them, and consistency with secondary objectives such as public recreation, aesthetics, protection of public safety, and groundwater recharge. It is usually submitted to regulatory officials for their review.

"Standard-based" design—Design of urban stormwater management facilities based on some specified set of regulatory standards. An example is the stipulation in local drainage policies that culverts for a given subdivision all be designed to pass the 10-year flood before road overtopping.

"Risk-based" design—Design of urban stormwater management facilities not only on the basis of local standards, but also on the basis of the risk (cost) of the flow exceeding a selected design. Virtually all stormwater management projects have some component of risk which is inherent in selection of a design return frequency. Risk may also account for special upstream or downstream hazards that would be posed by adherence to some recommended standard. For example, the designer of culverts in a subdivision might choose to upsize particular culverts from a 10-year to a 50-year basis to protect properties, or to make other provisions to secure emergency discharge capacity.

Multiple-purpose facility—An urban stormwater facility that fulfills multiple functions such as enhancement of runoff quality, erosion con-

Breckenridge, Colorado—Small lake immediately adjacent to a hotel. The impoundment created by the low dam in the foreground serves as a magnet for tourists.

trol, wildlife habitat, or public recreation, in addition to its primary goal of conveying or controlling runoff.

Conveyance structure—A pipe, open channel, or other facility that transports runoff from one location to another.

Drainage criteria—Specific guidance provided to the engineer/ designer to carry out drainage policies. An example might be the specification of local design hydrology ("design storm").

Finally, the reader should remember that while this is a manual of practice, it should not be viewed as guidance that will apply forever (or even until the next update), but rather as the best current information available. The technology and practice of urban storm drainage is changing so rapidly that the engineer will have to adjust his thinking, and his plans and designs, as criteria, techniques and materials change.

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Chapter 2

FINANCIAL, LEGAL AND REGULATORY CONCERNS

I. INTRODUCTION

The storm drainage component of the urban infrastructure has historically received less attention than other elements such as water supply, streets, airports, and wastewater collection and treatment. Out of sight, out of mind. As urbanization has intensified in the last 40 years and extended into previously rural areas, the consequences of poor stormwater management have become obvious. Urbanization causes significant increases in runoff peaks and volumes. If these increases are not anticipated and if adequate facilities are not provided, relatively small storms can flood streets, interrupt traffic, and cause property damage. Large storms can flood creeks and rivers, reclaim floodplains, cause significant property damage, and threaten lives. Construction of urban stormwater systems often has lagged behind other urban infrastructure systems.

The importance of good stormwater management has been acknowledged by planners and engineers, and more importantly by those responsible for urban development. Important factors have included the mortgage insurance programs of the U. S. Department of Housing and Urban Development that stimulated the adoption of drainage design standards for subdivision development, creation of the Federal Flood Insurance Program that has provided impetus for most local governments to adopt and enforce floodplain regulations, and court decisions that have placed responsibility for drainage problems on those who could be identified as causing the problem. Many local governments have adopted storm drainage criteria, and some have begun charging fees for providing a drainage service. Most recently, the adoption by Congress of the Water Quality Act of 1987 will require the nation's larger urban areas to prepare and implement stormwater quality management plans.

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It has become essential for the engineer to understand the financial, legal, and regulatory requirements and design constraints. Since these issues can dominate the drainage system process, they must be addressed in concert with the technical aspects of the assignment.

The responsibility for management of the urban storm drainage system is typically not clearly defined, although some generalizations can be made. One problem has been that poor drainage control is obvious only during a time of heavy rainfall, whereas other urban systems such as water supply, transportation, and sewage disposal are more obvious when inadequate or missing. A subdivision cannot be built without streets, for example, but the lack of good drainage may not be so obvious, until storm runoff problems occur.

The responsibility for urban drainage rests primarily at the local level, and drainage facilities should be an inseparable part of urban infrastructure planning and design. The private sector generally is called on to provide storm drainage as a part of new developments (although operation and maintenance remain local government responsibilities). Federal and state agencies have tended to have greater influence and involvement with larger drainage systems, and increasingly are setting minimum requirements (or superseding local criteria), particularly in relation to floodplain management and water quality. Local governments, however, usually end up with the financial responsibility and liability for the entire system.

The importance of urban drainage system maintenance cannot be over-emphasized. The funding of maintenance is a problem, however, and in many communities deteriorated public facilities threaten the provision of basic community services, including flood protection (Choate and Walter 1983). A local tax base usually is necessary to support maintenance on a continuing basis to keep facilities in operating condition. Storm drainage facilities on private property are generally the responsibility of the owner. "Responsibility" involves financial, legal, and regulatory concerns, and the remainder of the chapter will address these three areas.

II. FINANCIAL

The most important financial considerations are who pays and what is the source of funds. Economic evaluation of master plan alternatives and preparation of construction cost estimates are covered in Chapter 8. Basic funding sources include (1) financing from outside the local area such as federal and state assistance; (2) financing from within the local area, either directly from local governments, from improvement districts, or from private interests such as developers. Financing can be difficult to obtain, and many well-conceived design and construction programs are never implemented because of the lack of funding. Operation and maintenance costs must be considered, and there must be a commitment to maintenance by local government or some other entity

that can be held responsible and that has the necessary financial capability.

The review and development of financing methods for a project or community generally involves engineers as a part of the team. They are involved in preparing preliminary plans and cost estimates, defining drainage basins and hydrologic responses, identifying floodplains, assessing water quality impacts, and defining frequency and extent of flooding problems. This section reviews available financial options, which should not be considered in isolation, but viewed as a possible package of several options. The feasibility of different financing approaches varies widely from one area to another, and the nuances of each situation must be carefully considered, particularly in relation to relevant statutory authorities.

A. State and Federal Sources

Funding support for design and construction of urban storm drainage systems is limited at the state or federal level. Flood control, which deals with major drainageway or river flooding, receives some support from federal programs of the Corps of Engineers (COE) and the Soil Conservation Service (SCS). Little other funding is available, except from sources such as revenue sharing or block grants, for addressing storm drainage problems.

The following describes funding opportunities that existed as of the early 1990's. Regulations implementing federal and state programs are constantly changing, and federal and state sources of funding change as programs are modified, eliminated, or added.

1. U.S. Army Corps of Engineers (COE)

The COE has been in the flood control business essentially since 1936 (Public Works Historical Society 1988), and has several programs that can aid in addressing urban flood control and erosion problems (U. S. Army Corps of Engineers 1990). The COE is authorized to provide technical assistance to local communities and states to support their efforts to control flooding and reduce erosion, and also is authorized to construct small flood control projects without specific Congressional approval. Large flood projects require the authorization of Congress.

There are several technical assistance programs. The Shore and Streambank Protection program (Section 14, Flood Control Act of 1946, as amended) can help design projects to prevent or repair damages that occur from shoreline and stream bank erosion. Floodplain management services (Section 206, Flood Control Act of 1960, as amended) are available to help local communities identify flood hazards and plan for wise use of floodplain lands. The Channel Renovation Program (Section 942, Water Resources Development Act of 1986, as amended) can provide designs, plans, specifications, and other technical assistance for renovation of navigable streams and their tributaries.

Construction programs include the Small Flood Control Projects programs (Section 205, Flood Control Act of 1948, as amended) which enables the COE to construct small flood control projects. This program focuses on solving local flooding problems in urban areas. The Emergency Streambank and Shoreline Protection Program (Section 14, Flood Control Act of 1946, as amended) allows the COE to construct emergency streambank and shoreline protection to prevent erosion or flooding from damaging highways, bridge approaches, hospitals, churches, schools, and other non-profit or public facilities, such as wastewater treatment facilities. The COE can clear stream channels to increase flow capacity, decrease flooding, and reduce damage from debris carried by flood flows through the Channel Clearing for Flood Control Program (Section 208. Flood Control Act of 1954, as amended). Finally, the Congress can authorize and fund individual COE projects, but this is a lengthy process, with the average project taking over 20 years to get under way.

The Water Resources Act of 1986 provides for COE and local sponsor cooperation, and defines minimum financial requirements for local sponsors. Local sponsors are afforded more active involvement in defining and planning the projects, but a greater local financial commitment is required.

2. *Soil Conservation Service (SCS)*

The SCS is authorized to provide technical and financial help to local organizations for floodplain management assistance, watershed protection, emergency flood relief, and flood warning assistance under Public Law 83-566, commonly called the Small Watershed Program (Soil Conservation Service 1984). While this program is agriculturally oriented, in some cases it is possible to address urban flooding and erosion control problems.

Projects are initiated by local sponsors, which can include municipalities, counties, and flood control or other special districts, who submit an application to the state agency designated to approve watershed applications. The applications are reviewed by the State and the SCS. If the project has a high priority the SCS Chief can authorize planning assistance, and the SCS and sponsors can proceed with project planning. Depending on the amount of federal assistance involved, construction approval of a project is made by the SCS state conservationist, by the SCS Chief, or by Congress. The SCS pays the full cost of construction for flood protection, and shares the cost for other purposes. Major obligations of local sponsors are to acquire lands, easements, and rights-of-way, and to share in certain construction costs.

3. *Environmental Protection Agency (EPA)*

There has been little EPA funding for urban storm drainage in recent years, except in limited amounts for implementation of projects and programs established under the nonpoint source pollution control pro-

gram (Section 319 of the Federal Water Quality Act (FWQA) of 1987). The FWQA authorized federal funds to capitalize state revolving fund (SRF) programs and to provide a transition from the former Federal Construction Grant Program to a new approach for assisting communities in construction of publicly owned treatment works (POTWs). From the SRF, states can provide loans and other types of financial assistance (but not grants) to local communities and inter-municipal and interstate agencies, for the construction of POTWs and implementation of nonpoint source programs and projects. Some urban stormwater quality control projects may qualify for SRF assistance.

4. *Community Development Block Grant Program*

The Community Development Block Grant (CDBG) Program is a grant program to assist local communities with specific projects, funded through the Department of Housing and Urban Development (HUD). There are two types of CDBG programs. One provides annual CDBGs on a formula basis to entitled communities to carry out a wide range of community development activities directed toward neighborhood revitalization, economic development, and improved community facilities and services (U. S. Department of Housing and Urban development 1989). Metropolitan cities and urban counties (generally those with populations in excess of 50,000) are entitled to receive annual grants. Entitlement communities develop their own programs and funding priorities.

The other CDBG program is a non-entitlement program for states and small cities. Grants are awarded competitively based on established selection criteria. Each state has the option to administer the block grant funds provided for its non-entitlement areas (most states exercise this option). Typically grants are awarded competitively based on established selection criteria.

Drainage system projects are eligible for CDBG funds if they meet basic program requirements. Local matching funds are not required, however the competition for these funds is intense, and they will probably become more difficult to obtain in the future.

5. *Federal Emergency Management Agency (FEMA)*

FEMA is the designated leader of inter-agency task forces that are formed to provide assistance following emergencies or major disasters. This assistance can include low interest loans to the private sector and partial grants to local governments for repair or replacement of damaged public facilities. (Federal Emergency Management Agency 1989)

A major FEMA activity is the administration of the National Flood Insurance Program (NFIP), which is also discussed under the Regulatory section of this Chapter. FEMA also has limited funds available for the purchase of flood-damaged property under the provisions of Section 1362 of the Flood Disaster Protection Act. If a structure has been damaged repeatedly by flooding it may qualify for acquisition under this program.

6. *National Weather Service (NWS)*

The NWS has authority to assist local interests to develop and implement local advance flood warning programs.

7. *State Funds*

It may be possible in some states to obtain state funding support for drainage and flood control projects, although states are generally poor sources. Assistance may take the form of planning advice, floodplain delineation, flash flood and contingency planning assistance, revolving loan funds, or construction of facilities.

B. Local Sources

Traditionally, urban stormwater drainage has been primarily the responsibility of local government. Although there is increasing regulatory pressure by state and federal agencies to resolve urban drainage issues, the burden of financing urban drainage programs will continue to fall primarily on local governments and private developers. Demands and requirements for more involved and complex urban drainage programs are increasing, which is causing the costs associated with providing and maintaining urban drainage systems to rise.

Financing needs can be categorized into two basic areas: (1) existing development, or problems needing remedial treatment, and (2) new development, or situations offering the opportunity to prevent future problems. Financing needs include both annual operating costs (including planning, administration, regulatory enforcement, maintenance, and operations) and capital improvements. Some practices for financing urban storm drainage projects and programs are listed in Table 2.1. In most cases the sponsors will use a combination of these funding sources. Some of the approaches lend themselves to financing annual operating costs, and some to capital expenditures.

TABLE 2.1. Methods of Financing Urban Storm Drainage Programs

Development Status (1)	Source of Funds (2)	Primarily O&M or Capital (3)
Existing developments	General tax fund	O&M or Capital
	Service charge or fees	O&M
	Special assessment	Capital
	Bonds	Capital
	Private funds	Capital
	Stormwater Utilities	O&M or Capital
New developments	Developer fees	Capital
	Developer provided facilities	Capital
	Dedications	Capital
	Floodplain regulations	Capital

1. General Tax Revenues

All local governments support their operations with basic tax revenues, including property taxes, sales taxes, fees, licenses, etc. There is always competition for these funds, and it is difficult to raise large amounts of capital for drainage improvement and maintenance. When flooding problems exist on major drainageways in developed areas, structural improvements, rights-of-way, engineering, and construction are particularly costly. General fund revenues usually are best suited for financing operation and maintenance, and less suited for funding large capital projects. There are exceptions. For example, a portion of a sales tax can be dedicated to capital projects including drainage, or funds can be annually escrowed into a sinking fund to pay for an improvement. General funds are in most cases the only source of money for planning, plan review, inspection, mapping, and similar activities.

2. Dedicated Ad Valorem Taxes

Most flood control districts and a few city and county governments have the authority to levy taxes on improved property specifically for storm drainage and flood control. The state of Illinois recently passed legislation allowing a county-wide tax for stormwater management in several urbanized areas. While ad valorem taxes are a significant source of revenue, the impetus for many jurisdictions to treat stormwater as a "utility" reflects the desire to allocate program costs on criteria more directly related to a property's contribution of runoff than to assessed valuation. There is no clear correlation between property value and contribution of runoff to the drainage system, whereas a strong correlation exists between the amount of impervious surface and impacts on the stormwater system. Increased equity is therefore a principal element in the decision to move toward stormwater service charges, and away from ad valorem taxes.

3. Service Charges or Fees

Many local governments are turning to service charges or fees for financing urban drainage programs because of competition for limited general fund tax dollars. The fee should be related to service provided, the most common basis being area of impervious surface. The principle is that each property owner pays a fee for the service of handling the drainage originating from that property. This is fair because uphill properties generate the same runoff as similarly developed downhill properties. Funding does not fall entirely on those who experience flooding problems, but is distributed equitably to all those who contribute to the problem. Such fees are particularly useful for industrial facilities, which discharge large amounts of water to the urban stormwater system.

Service charges have been adopted by large and small cities, as well as counties. In the Denver metropolitan area four cities have a drainage

service charge. They are Denver (509,800), Boulder (80,400), Aurora (223,200) and Littleton (31,100). Drainage fees involve billing each individual property in the city. In some cases, such as Boulder and Littleton, the drainage fee supports a utility.

Some other cities in the United States with service charges include Tulsa, Oklahoma; Bellevue, Washington; Tampa, Florida; Cincinnati, Ohio; and Louisville, Kentucky. Generally, a uniform method of determining fees is applied throughout the service area, although fees will differ for various classes of property, usually based on the area of a property and its percentage of impermeable area.

There are numerous combinations and variations of stormwater service charge rate structures available (and that have been upheld in the courts). The three approaches most often used are:

(a) Amount of Impervious Surface. Under this approach, rates are set in direct proportion to the measured, estimated, or assumed extent of impervious area for each parcel of land.
(b) Density of Development. Rates are determined by a runoff coefficient which is deemed appropriate for the type of land and the nature of the improvements to each parcel.
(c) Flat Fee. This mechanism uses a constant or uniform fee for each property within pre-existing classes, or can be applied on a community-wide basis. Flat fees are used mainly because of their administrative simplicity.

Charges for stormwater management reflect a rationale that those who contribute to stormwater problems should bear the costs of mitigative services. This approach is regarded by most administrators and the courts as an appropriate technique for financing stormwater programs. Public acceptance is critical, however, and public awareness and problem recognition are the most important elements in the successful implementation of service charges.

Generally, operation and maintenance costs are paid from drainage fees. Drainage fees can also be used to finance capital projects through revenue bonds or on a pay-as-you-go basis. Capital needs vary from basin to basin and it would be most equitable to have the portion of the drainage fee for capital projects reflect the specific needs of the basin. Some disadvantages of drainage fees can be the need to obtain large amounts of detailed data about the location and size of properties and their impervious areas, the need for a system to bill each individual property, and the resistance of many property owners, particularly those who live uphill and do not have a drainage problem (although they contribute to drainage problems and benefit from improved urban drainage).

The most important advantage of a fee is that a dedicated and dependable source of funding can be made available, enabling the local government to think and plan for the long term. The drainage system can be viewed as a utility whose components can be considered together in their entirety. Fees also provide dependable funding for operation

and maintenance, a critical part of an effective urban drainage program. Service charges are coming into more general use as urban drainage system needs become more pronounced and as more pressure is placed on general revenue sources. Since a drainage fee is not considered a tax, it can be charged to all landowners, public and private, from whose property storm waters flow. This includes traditionally nontaxable institutions such as churches, colleges, government facilities, and even state and local highway departments in some cases.

4. *Special Assessments*

A special assessment is a mandatory charge on selected properties for an identified improvement which benefits the property owners and which is undertaken in the public interest. Special assessment projects may be undertaken by general purpose governments or special purpose districts. The authority for local governments to levy special assessments is derived through enabling state legislation. The requirements and procedural details for their establishment vary from state to state, and from city to city.

Special assessments can generally be initiated in one of the following ways:

(a) By local government legislative body (council, commissioners, aldermen, etc.) action with consent of property owners, usually expressed in the form of a petition.
(b) By local government action, which may be stopped by opposing petition or remonstrance.
(c) By local government action without property owner consent and without possibility of remonstrance.

The benefit that accrues to each property by virtue of the project provides the foundation for levying a special assessment. The amounts assessed must be proportional to and not more than the benefits received, and must not exceed the cost of the project. Problems with special assessments include determining who is specially benefited, and determining the amount of special benefit to be received by each property owner. Benefits may include handling the discharge of surface waters from uphill properties through lower properties in a quantity greater than would naturally flow because the uphill owner made some of their property impermeable; adaptability of property to a more profitable use; alleviation of health and sanitary hazards; reduction in property maintenance costs; increase in convenience or decrease in inconvenience; and recreational improvements. As with service charges, special assessments may be made on properties that are normally non-taxable.

5. *Bonds*

Bonds are not an additional source of revenue per se, because they create an equivalent liability that must be met from pledged future

revenues. Bonds provide a mechanism for local governments to borrow capital needed to finance public works improvements. They generally must be authorized by a vote of the electorate.

There are two basic types of bonds, general obligation (GO) bonds and revenue bonds. General obligation bonds are backed by the full faith and credit of the unit of government issuing the bonds. They may in fact, be repaid from fees, but if the fee revenue is inadequate, general tax revenues would have to be used. Revenue bonds are repaid entirely from revenues such as services charges or fees, and are not guaranteed by the issuing entity. The advantage of GO bonds is that their interest rates (and the amount required as "reserve") are lower.

6. Stormwater Utilities

Stormwater utilities are becoming increasingly popular as a means of financing and managing stormwater systems. The concept is the same as for other common utilities, such as water and telephone, which provide a service to their customers. A stormwater utility constructs capital improvements and operates and maintains the facilities in its system. Their operations are financed by regularly billing their customers for these services. Fees may be based on various criteria, including those enumerated in preceding sections. Administrative appeals and a formal variance procedure are typically accommodated. Utilities are particularly attractive because they provide a single unit with its own staff, regulations, budget and sources of revenue which can focus on solutions to drainage problems.

7. Development Fees and Developer-Provided Facilities

Developers have typically been required to provide on-site drainage facilities such as curbs and gutters, inlets, storm sewers, and detention ponds, however developer-provided major system facilities are less common.

A method of financing flood management projects impacting an entire basin is to spread the cost of required facilities over the entire basin. The rationale is that a development should finance those regional improvements that are necessitated by the cumulative development. A method of accomplishing this is with a development fee—a unit fee based upon acreage involved for all developments in the basin. The amount of the fee would depend on the cost of facilities, including right-of-way required in the basin, and the fee would vary from basin to basin. A basin development fee should be charged only for facilities required because of development, and not to finance improvements required to solve previously existing problems.

A master drainage plan defining needed facilities for each basin must be developed to determine the amount of the basin fee. One of the difficulties with drainage development fees is that an improvement defined in the master drainage plan may be needed before adequate funds are available in the basin fund to finance the project. This may

require governments to provide or loan the basin fund enough monies to build the necessary facilities. Depending on future development in the basin, local governments may or may not recover their money.

8. Fee in Lieu of On-Site Detention/Retention

In-lieu-of fees are derived from system development charges specific to stormwater management. These fees can be either a regulatory requirement or a development option that affords the opportunity to construct on-site detention/retention facilities in accordance with established design criteria (i.e. local or county), or to pay a fee into a fund dedicated to the construction of a regional detention facility serving multiple properties. This approach is typically authorized within a context of promoting the siting and construction of more regional, as opposed to on-site, detention/ retention facilities. This objective is consistent with the intent of regional detention ordinances, which have proven effective as a vehicle to guide development patterns within a watershed, and as a tool to encourage comprehensive stormwater planning.

The problems most frequently encountered with fee-in-lieu construction involve cash flow and construction timing. The customary fee for a single property or development is rarely large enough to fund the construction of a regional facility. Therefore, either multiple developments must occur simultaneously or, more realistically, the project must be initially funded from alternative sources.

9. Plan Review and Inspection Fees

These fees are intended to recoup the expense of examining development plans to insure consistency with comprehensive or master plans, and to insure that design and construction standards are met. These fees are **not** designed to be primary revenue-generating sources. In theory, a detailed cost-accounting system can determine the actual costs of providing engineering review and field inspections/certifications. In practice, however, most drainage authorities apply a fee based on an average of their total costs. Four fee structures are commonly applied:

(a) Fees based on a flat rate for all projects reviewed or inspected, regardless of size or complexity.
(b) A variable or sliding-scale fee based on the size of development.
(c) A variable or sliding-scale fee based on the permitted construction cost of the development or project.
(d) A fee based on the fixed and variable costs to provide the review and inspection service. The fixed portion is usually a statistical estimation of the administrative costs to provide the service, while the variable portion reflects the actual time and materials required to perform the plan reviews and inspections.

One of the keys to a successful regulatory program is consistent enforcement of development/construction controls. By implementing a

plan review/inspection charge based on the actual cost of providing the service (as in the case of code enforcement staffing), the program can both enhance the development/construction review process and avoid passing the costs of these direct services back to the general public.

10. Dedications

When new developments infringe on a floodplain the developer must consider the expense of meeting floodplain requirements. An alternative to structural improvements is to avoid the floodplain. The developer must compare the cost of structural improvements and potential revenues with the cost of not developing the floodplain. If the cost of development is excessive, the developer may choose to leave the floodplain alone and dedicate that area to a public entity (thereby providing valuable open space). Some jurisdictions promote the dedication of floodplain areas by granting density credits for land so dedicated, and by allowing cluster development on the remainder of the tract. It should be noted, however, that many jurisdictions still view floodplains as less desirable for recreation and open space, and resist the dedication of floodplain lands as the developer's open space requirement. Local governments should be aware that urbanization may accelerate erosion on "natural" drainageways, and that erosion control measures may be required in the future.

III. LEGAL

Legal considerations are often a determining factor behind drainage decisions. The engineer must appreciate drainage law and the limitations it may impose on design, and must think in terms of public safety and welfare and his professional liability throughout the course of a design effort. Designing and installing drainage systems that conflict with established drainage law or statutes, or that are inconsistent with good engineering practice, must be avoided because the engineer may be held liable for flood related damages that may occur. The information in this section is based on the Denver Urban Drainage Criteria Manual (Urban Drainage and Flood Control District 1984), Wright (1982), APWA (1981), and the Urban Land Institute (1975).

A. General Rules of Law

The "law" is derived from judicial decisions and statutory enactments. Over the years, two general rules of "law" have developed with respect to surface waters. More recently, judges have modified the original rules.

In the early years of rural America, the "common enemy" doctrine prevailed. Surface waters were considered a common enemy that each landowner could deal with as he pleased, by repelling, diverting or even retaining them. However, in some jurisdictions, judges applied a

"civil law" rule which places a servitude upon the lower land in favor of the upper or dominant landowner to receive surface water in its natural way of draining. The passage of this water is not to be obstructed by the servient owner to the detriment of the dominant owner. This rule has been modified over the years to give some protection to servient owners and now follows the premise that "natural drainage conditions may be altered by an upper landowner provided the water is not sent down in a manner or quantity to do more harm than formerly."

The approach followed by judges today in resolving drainage disputes in many jurisdictions is called the "reasonable use" rule. Each property owner, including a municipality, can legally make use of his land, even though the flow of surface water is altered and causes some harm to others. However, liability attaches when his harmful interference with the flow of surface water is "unreasonable." Whether a landowner's use is unreasonable is determined by a nuisance-type balancing test involving three questions:

(a) Was there reasonable necessity for the actor to alter the drainage to make use of his land?
(b) Was the alteration done in a reasonable manner?
(c) Does the utility of the actor's conduct reasonably outweigh the gravity of harm to others?

The North Carolina Supreme Court in 1977 traced the evolution of common-law rule to the civil law, and then finally to the reasonable use rule by stating:

> "It is no longer simply a matter of balancing the interests of individual landowners; the interests of society must be considered. On the whole the rigid solutions offered by the common enemy and civil law rules no longer provide an adequate vehicle by which drainage problems may be properly resolved."

As urbanization has taken place, drainage disputes have typically involved several parties, not just one upper owner and one lower owner. The "interests of society," referred to by the North Carolina Court, have put municipalities into the issues of drainage. In 1969 the Colorado Legislature recognized the issue of urban drainage problems when it created the Urban Drainage & Flood Control District to address flooding problems in the Denver metropolitan area (Urban Drainage and Flood Control Act 1969). The Colorado General Assembly in its legislative declaration stated that:

> "The necessity of this article results from the large population growth in the urban area included by this article within the district constituting a major portion of the state's population, from the numerous capital improvements and large amount of improved real property

situated within such urban area, from the torrential storms occurring sporadically and intermittently in the urban area and other areas draining into such urban area, from the increasing danger of floods therein and the resultant risks to the property and to the health and safety of the persons within the urban area, . . . from the fragmentation and proliferation of powers, rights, privileges, and duties pertaining to water, flood control, and drainage within such urban area among a substantial number of public bodies, and from the resultant inabilities of such public bodies to acquire suitable capital improvements for the alleviation of such dangers and risks."

Many courts have recognized that:

(a) Since surface water flows downhill, don't try to stop it.
(b) He who casts more water downhill (by urbanization, for example) than would naturally flow, must provide for it.
(c) A municipality is generally treated like a private party in drainage matters.
(d) If you are not going to maintain it, do not build it.
(e) A municipality may be liable for issuing permits to a developer which increase flooding of downstream property.

And some legislatures have enacted laws that:

(a) Require runoff caused by developments in excess of historic flows be detained.
(b) Define "special benefits" for the purpose of determining special assessments to include "accepting the burden from specific property for discharging surface water into servient property in a manner or quantity greater than would naturally flow because the dominant owner made some of this property impermeable."
(c) Permit local governments to charge service fees to users of drainage facilities.
(d) Require or allow local governments to identify, designate, and regulate the 100-year floodplain.

B. Liability Issues

Potential liability is becoming of major concern to engineers, their employers, and their clients. The following discussion addresses some potential liability issues, however the engineer must rely on his own common sense, professional competence, and judgment to limit his exposure to liability. Expert legal advice should always be sought if there are questions or concerns.

1. Ordinary and Extraordinary Floods

Whether or not defendants were liable for their actions has often depended on whether the flood was determined to be "extraordinary" or "ordinary." The extraordinary flood was considered to be an act of

God and a defense against liability. Even early on, however, some courts did not recognize this and would not absolve a defendant when damages occurred due to an act of God, where the defendant's negligence was the proximate cause of injury. With the advances in hydrology and meteorology in recent years, courts are now able to base their decisions on scientific data presented to them by expert witnesses, with the result that it is virtually impossible to rely on the Act-of-God defense. For example, standard hydrologic techniques permit quantification of large floods such as the standard project flood (SPF) commonly used by the COE or the PMF, and dams in urban areas can and should be built to safely handle this extremely large event without failing. Dam design criteria are generally established by the states, and any such criteria must be carefully followed by a design engineer.

Other adjectives used by the courts are "unprecedented" and "unanticipated." In situations where the court may feel that the defendant's actions were reasonable and not negligent the "unprecedented flood" defense may still be used. One example is where a city maintained a natural waterway as a drainage ditch. A storm "in excess of 100 years" caused water to back up and damage plaintiff's property. The court found that the city had properly maintained the drainageway and held the city not liable because the storm was unprecedented. However, in another case where the flow was increased by upstream urbanization, and the city could have avoided the damage by simply replacing an inadequate culvert in a highway, the court held that the flood was not extraordinary but could have been anticipated.

Generally, an engineer should not assume that he will be able to use the "Act-of-God," "unprecedented," or "unanticipated" defense. Local flood or rainfall history may indicate flow peaks or rainfalls of comparable or greater magnitude than the event that is considered "unprecedented." When liability for flood damages becomes an issue it may be necessary for the engineer to demonstrate that he (1) practiced without negligence; (2) adopted the maximum design frequency specified by the regulation; (3) followed applicable drainage standards and criteria; (4) researched historic flood information; and (5) checked the design for events greater than the design flood.

2. *Public Liability*

A municipality is generally treated like a private party in drainage matters. Municipalities normally have no legal obligation to construct drainage improvements, unless public works such as schools or roads alter the drainage and require drainage improvements to prevent potential damage to others. Municipalities can be held liable, however, for negligent construction of drainage improvements as well as for their negligent maintenance and repair. In general, in the absence of negligence, a municipality will not be held liable for damages caused by overflow of its sewers or drains occasioned by extraordinary, unforeseeable rains or floods.

Municipalities can be held liable, however where they:

(a) Collect surface water and cause it to flow onto private property where it did not formerly flow.
(b) Divert surface water in such a way that it causes damage on private land where, but for the diversion, it would not go.
(c) Fill, dam, or otherwise divert a stream so that it flows onto the land of another.
(d) Fail to construct or have constructed drainage outfall facilities to carry developed flow from new subdivisions located in the upstream areas of a basin.

A municipality is also liable if it fails to provide a proper outlet for drainage improvements constructed to divert surface waters, or if it fails to exercise ordinary care in maintenance and repair of drainage improvements. Whether or not public entities have a "continuing duty" to upgrade drainage facilities in response to upstream development is not clear (in the law). The engineer should recognize, however, that design standards generally become stricter with time, and that facilities may need to be upgraded.

The courts have been reluctant to find liability when a municipality merely issues a permit to build, unless an ordinance or statute imposes a duty on a municipality to prevent or protect land from surface water drainage. The liability, if any, would be against the developer or builder. On the other hand, some courts are finding municipalities liable in situations where the municipality issued permits for development that caused injury to other property owners. An example is where a municipality granted a permit to develop an industrial complex that increased flooding on downstream property that had flooded previously. The municipality was held liable for granting the permit, had to pay damages, and was enjoined from issuing any more building permits.

There seems to be a trend toward imposing a greater burden of responsibility on municipalities for the drainage consequences of urban development. Municipalities should proceed with caution when issuing permits for development where the adverse impacts on drainage can be determined and foreseen.

3. *Other Liability Considerations*

Examples of questions or concerns of which the engineer should be aware are listed below. No opinions or answers are provided, but if these or similar concerns arise the engineer should seek legal advice.

(a) Should safety racks be installed at culvert entrances? At outlets?
(b) What constitutes a "dangerous" hydraulic structure from the standpoint of public safety?
(c) How much can and should the engineer disrupt an area's drainage "status quo" with new facilities? For example, a Federal District Court in Cheyenne, Wyoming found that a culvert with a capacity exceeding that required to pass the 100-year flood constituted an inadequate re-

placement structure for a far larger bridge, because reliance on the bridge had been established by upstream property owners. The bridge had been in place since 1910.

(d) To what extent are land developers, and their engineers, liable for downstream erosion, sedimentation and water quality problems?

(e) To what extent should buildings be allowed to encroach on floodplains downstream from dams?

Local case law should be searched to see if questions such as these have been litigated where the engineer is practicing, if there is a possibility that they could become an issue.

4. *Limiting Liability*

There are a number of common-sense rules that can be adopted to minimize potential liability. These include:

(a) Rigorous in-house review of all assumptions and calculations.

(b) Peer review by independent parties.

(c) Meticulous record-keeping, including calculations, telephone conversations, meeting notes, documentation of regulator/client decisions, contracts, etc.

(d) Step-by-step review and endorsement of design assumptions by review authorities.

(e) Searching for unusual and unforeseen problems that can arise, such as failure of upstream structures or changed geotechnical conditions.

(f) Avoidance of practices that "beg" litigation, such as failure to comply with contract provisions.

IV. REGULATORY CONCERNS

Several regulatory programs have significant impacts on urban stormwater management. Local governments generally enforce regulations by authority granted to them by state enabling legislation. There are some programs that are either required or enforced by the federal government, such as Section 404 of the Federal Water Pollution Control Act which includes the dredge and fill material permit program and the wetland protection program.

A. Floodplain Regulation

The purpose of floodplain regulation is to manage, not prevent, development within a defined floodplain (usually the 100-year floodplain) so as to preclude or mitigate future flood damages. Legal justification for floodplain regulation is the health, safety, and welfare of the public. Floodplain regulation should not preclude all uses of the floodplain, and should not be used to keep floodplains in an open undeveloped state.

With some exceptions, such as New Jersey, the authority to regulate

floodplains is generally delegated by state legislatures to local government. Some states have minimum requirements; others do not. In a few states local governments must regulate floodplains or the state will do it for them.

A basic requirement for a successful floodplain regulation program is accurate delineation of the floodplain based on full development of the basin consistent with projected land uses. Detailed delineations are particularly important in urban and urbanizing areas because of the large number of residents, density of development, and quantity of drainage flows.

Floodplain regulations can take various shapes and forms, but owners/developers of floodplain lands have several basic choices. They can build outside of the floodplain and leave the floodplain alone. This alternative has a low capital cost, but less land is available for development. They can reclaim the floodplain by constructing a channel to contain flood flows. This alternative has a higher capital cost, but maximizes the land available for development. The flood fringe can be filled, which reclaims some of the floodplain but generally not as much as channelization. Other approaches also are possible.

Floodplain regulation is most effective in undeveloped floodplains, where the opportunity to control future development is the greatest. In urbanized floodplains the potential for flood damage already exists. Regulation, however, can prevent increases in the damage potential, and can have a positive effect on damage reduction where older buildings are being rehabilitated or the area is being redeveloped.

The National Flood Insurance Program (NFIP) has had a significant impact on floodplain regulation by local governments. This federal program makes flood insurance available at reasonable rates to individuals within communities that meet eligibility requirements by adopting and enforcing measures to reduce future flood risks to new construction in defined flood hazard areas. If local communities choose not to meet NFIP requirements, FEMA will prepare flood hazard boundary maps for the community, but will not make the federally subsidized insurance available, and will withhold federal financial assistance and support from the community for locations within identified flood hazard areas. For example, Small Business Administration, Veterans Administration, and Federal Housing Administration assistance, as well as other federal grants, loans, or guarantees, are prohibited within identified hazard areas unless the community participates in the program. Post-flood disaster aid is similarly reduced, which can be a serious loss to a community that suffers a flooding disaster.

Community participation in the NFIP is voluntary, although some states require NFIP participation as a part of their flood management program. For all practical purposes, however, the regulation of floodplains has become a matter of necessity for local governments rather than a matter of choice. Most local governments have taken the steps necessary to participate in the NFIP.

B. Section 404 Permits

Many engineers have heard (or asked) the question—"do we need a 404 permit for this project?" This refers to Section 404 of the Federal Water Pollution Control Act, as amended, which prohibits the discharge of dredged or fill material into the waters of the United States, including wetlands, unless a permit is obtained from the Corps of Engineers. Many urban drainage and flood control projects are impacted by 404 permit requirements since they deal with water and drainageways. It is therefore imperative that engineers be aware of 404 permit requirements and address the need for a 404 permit early in the planning or design process. The COE District Engineer should be consulted if any questions arise.

Some of the important definitions and requirements of the 404 permit process are discussed below. The COE has published summaries and brochures explaining the program. Specific requirements are set forth in the Code of Federal Regulations (33 CFR Parts 320 through 330 and 40 CFR Part 230).

The 404 permit program applies to "the waters of the United States," which are defined (33 CFR 328.3(a)) as:

(1) All waters which are currently used, or were used in the past, or may be susceptible to use in interstate or foreign commerce, including all waters which are subject to the ebb and flow of the tide.
(2) All interstate waters including interstate wetlands.
(3) All other waters such as interstate lakes, rivers, streams (including intermittent streams), mud flats, sand flats, wetlands, sloughs, prairie potholes, wet meadows, playa lakes, or natural ponds, the use, degradation or destruction of which could affect interstate or foreign commerce, and (a) which are or could be used by interstate or foreign travellers; (b) from which fish or shellfish can be taken and sold in interstate or foreign commerce; and (c) which are or could be used by industry engaged in interstate commerce.
(4) Tributaries of waters identified by items (1), (2) and (3) above.
(5) The territorial sea.
(6) Wetlands adjacent to waters defined above.

This definition obviously incorporates just about anything that is wet.

The definition of wetlands is also important. 40 CFR 230.41(a)(1) defines "wetlands" as those areas that are inundated or saturated by surface or groundwater at a frequency and duration sufficient to support, and that under normal circumstances does support, a prevalence of vegetation typically adapted for life in saturated soil conditions. Wetlands generally include swamps, marshes, bogs, and similar areas, though in arid regions they can also include washes and other areas that are not "wet" the year around.

The COE has limited the scope of the program somewhat by issuing nationwide permits for some activities which result in discharges into certain waters of the United States. If certain conditions are met, the

specified action can take place without the need for an individual or regional permit. The nationwide permit has all the restrictions and conditions set forth, and little or no paperwork is involved. It takes a relatively short time (0–20 days) to initiate an individual project. For example, a nationwide permit authorizes discharges into waters that are located above the headwaters. The term "headwaters" means that point on a perennial stream above which the average annual flow is less than 5 cfs. On an intermittent stream the "headwaters" is that point where 5 cfs is equalled or exceeded 50% of the time. Maps of the headwaters have been published and copies may be obtained from the COE. The use of this nationwide permit is limited, however, and is subject to future modification. In cases where the project impact exceeds one acre, written coordination with the COE is required. Other nationwide permits address storm drain lines, utility lines, and bank stabilization and maintenance activities.

Regional permits are a type of general permit and can be issued by a division or district engineer. A regional permit may require a case-by-case reporting and acknowledgment system. The regional permit will state what fill actions are allowed, what mitigation is necessary, how to get an individual project authorized, and how long it will take. The time required to initiate an individual project under a regional permit should be less than that for an individual permit. Projects that come under a regional permit must have minimal environmental impact, either separately or as a group.

An individual permit is for one action and the restrictions and conditions are tailored to the individual project. Extensive paperwork is involved and it usually takes 60 days or more to obtain. If there are any environmentally sensitive issues involved, or any objections to the work, it can take months or even years to obtain an individual 404 permit.

C. Erosion Control, Stormwater Detention, and Subdivision Ordinances and Codes

There are several areas of regulatory activity that are generally the responsibility of local government. They include erosion control ordinances, stormwater detention ordinances, and subdivision ordinances and codes. These activities are not influenced by federal programs in the same way that floodplain regulations are influenced by the National Flood Insurance Program. The authority which permits local governments to adopt and enforce such ordinances is derived from the state.

An important purpose of erosion control ordinances is to minimize the adverse effects of erosion and sedimentation from construction sites. The rapid conversion of land from natural or agricultural uses to urban uses may result in stripping the land of top soil, which can accelerate the processes of erosion and deposition. Construction-related sediment can be a source of pollution in downstream lakes, streams, ponds, and reservoirs and can result in large deposits on streets, and in drainage

channels, yards, etc. Detention ponds and small lakes can rapidly fill with sediment, necessitating expensive dredging to return them to their original effectiveness.

The greatest erosion potential occurs between the time when native vegetation is removed from a construction site and when construction is completed and restorative vegetation planted. Erosion control ordinances set forth practices, procedures, and objectives for the developer and his contractor during the construction period, which include limiting the extent of native vegetation that is disturbed, limiting the time during which construction takes place and the construction site is vulnerable to erosion, revegetating between phases of a construction project, and structural measures taken at the construction site such as reducing the velocity of runoff or providing sediment traps. Cities and/or counties commonly have erosion control manuals that provide guidance for meeting local requirements. In arid or semi-arid areas, urbanization often results in reduced long-term erosion, compared to predevelopment conditions. Some useful references on erosion and sediment control include Urban Land Institute (1978) and U.S. Environmental Protection Agency (1973). Many cities, counties and drainage districts throughout the U.S. prepared thoughtful erosion guidance and ordinances in the early 1990's.

Many local governments require that stormwater detention or retention basins be provided by developers. The purpose of such basins is to limit increases in the rate of runoff from impervious surfaces. Most detention and retention basins constructed by developers are in the 1–10 acre-foot capacity range. Larger detention facilities, generally built by public-sector agencies, can be "regional" facilities which can take the place of the smaller, randomly located developer-constructed facilities.

Detention and retention basins are generally designed to control runoff from the more frequent storms in urban watersheds, but they must be constructed to safely pass larger events. Design criteria vary considerably, and engineers must become familiar with local requirements. Basins can be classified as wet or dry. Dry basins hold back water only for short periods of time after storm events, whereas wet basins have a permanent pool below the level needed to store storm waters. Detention and retention basins may have other beneficial uses such as recreation, groundwater recharge, irrigation, industrial uses, water supply, sediment control, and pollution control. Local ordinances usually set forth criteria for the design of detention and retention facilities and require them to be constructed as a condition of development approval.

Detention and retention systems must not be considered substitutes for a drainage system. Detention basins have their greatest impact immediately downstream of the basin, but their effectiveness in reducing peak flows diminishes rapidly farther downstream. If downstream facilities are to be designed on the assumption that basins will be built, then a guarantee must be provided that all such facilities will in fact be constructed as designed, will remain in place, and will be effectively maintained. If this cannot be done, downstream facilities should be

designed as if the detention and retention facilities did not exist. Some ordinances will address the ownership and maintenance issue, and local governments will sometimes accept the ownership and maintenance responsibility. The transfer of maintenance responsibility to any group of private parties, such as homeowner's associations, is not recommended.

Subdivision ordinances guide the division of larger land parcels into smaller lots for development purposes. Most local governments of any size will have subdivision ordinances. The purpose of subdivision ordinances is to insure that development results in the provision of necessary basic facilities in a consistent manner. They control improvements such as roads, sewers, water, drainage facilities, recreation facilities, and dedication requirements. Floodplain regulations, erosion control ordinances, drainage ordinances, and detention ordinances may be included in a community's subdivision ordinance(s), or they may be separate from and in addition to those ordinances.

Building codes typically control building construction aspects such as the use of construction materials, but do not regulate the type or location of development. Building codes can include requirements that would tend to reduce flood damages. Examples are requiring suitable anchorage to prevent flotation of buildings during floods, requiring suitable locations for electrical outlets and mechanical equipment in flood-prone structures, restricting the use of materials that deteriorate when wet, and requiring adequate structural design to withstand effects of water pressure and flood velocities.

D. Stormwater Quality

Because of concerns over substantial pollution from nonpoint sources, the U.S. Environmental Protection Agency has undertaken an aggressive urban stormwater quality improvement program via the National Pollutant Discharge Elimination System (NPDES) permitting program under Sections 402 and 405 of the 1972 Federal Water Pollution Control Act (FWPCA) and 1987 Clean Water Act (CWA). Although the NPDES has traditionally focused on the reduction of pollutants from point sources (i.e. wastewater treatment facilities, industrial facilities), the current program will require permits for stormwater discharges.

Initially, this program has targeted larger American cities (those with population of over 100,000) and most categories of American industry. Smaller cities are required to obtain permits after October, 1992. Cities and industries are required to address such issues as: physical characteristics of storm drainage facilities; water quality sampling during "dry flow" and "wet weather" conditions to characterize runoff water quality; identifying "illicit discharges" and taking the steps necessary to remove these discharges from storm sewers; long-term mitigation planning and other activities. Traditionally, flood hazard reduction has been the driving force behind the implementation of storm drainage facilities. These regulations, however, in conjunction with a growing

public awareness of the extent to which urban runoff degrades receiving water quality, have increased interest in planning and implementing systems that provide both runoff quantity **and** quality management.

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Chapter 3
SURVEYS AND INVESTIGATIONS

I. INTRODUCTION

The purpose of surveys and investigations is to provide basic data for the replacement or upgrading of existing drainage facilities or for the design and construction of new storm drainage systems. Survey needs vary depending upon the stage of design, availability of reliable information, soil and groundwater conditions, requirements of agencies having jurisdiction over the project, and desired accuracy of hydraulic and water quality calculations. The engineer should approach surveys and investigations in a careful manner, and must learn to decide how much information is actually relevant and essential to the decisions that he will make. All subsequent work hinges on the accuracy, thoroughness and timeliness of field and office data.

Basic surveying theory and methods will not be discussed here. Rather, the emphasis is on the kinds of information and data typically required for urban stormwater management systems. Note that information should be collected both for existing and projected land use and regulatory conditions.

II. DEFINITIONS

The term "surveys" refers to the process of collecting and compiling information necessary to develop any given phase of the project. Surveys can occur either in the office or in the field. The office survey may include observations relating to general conditions affecting a project, such as historic, political, physical, fiscal, and others. The field survey may include the measurements necessary for the engineering design, as well as personal observations of the drainage area, present drainage facilities, geologic characteristics, and existing occupancy and improvements that merit particular attention. Extensive documentation of the

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study area with photographs is encouraged during field surveys. The designer is urged to spend as much time as possible in the field prior to undertaking even the most rudimentary design, as there is no substitute for first hand observation, especially during rainfall events (U.S. Department of Transportation 1980). While the term "investigations" is often used interchangeably with "surveys," this manual uses it to mean the process of assimilation and analysis of data produced by surveys.

III. MONITORING AND DATA COLLECTION

On small catchments, and where detention/retention storage and water quality are not design issues, use of the rational method and/or published rainfall data may be acceptable (for further discussion, see Chapter 5). If this is not the case, however, the engineer may be faced with the problem of collecting the necessary and appropriate data on which to base plans and designs.

Most analysis for stormwater planning and design is now done using mathematical models on digital computers, and hydrologic data are needed to calibrate and run these models. As will be pointed out in later chapters, the return frequency of runoff from a given rainfall event will normally be different from the return frequency of the rainfall itself. The engineer must then decide whether to use synthetic rainfall data and to rely on his model to generate necessary runoff data, or to collect rainfall and runoff data to calibrate and verify his models and provide a design basis that more nearly approximates the local situation (Huber and Dickinson 1988).

Given the availability of rainfall data in computerized form in the data bases of the National Weather Service (also available commercially), it is difficult to justify not making at least a cursory analysis of rainfall data from a local (or the nearest) weather station to develop some understanding of the statistical aspects of the precipitation regime, and to identify significant historical rainfall events. It should also be possible to identify rainfall events that produce significant runoff, either by historical observation or by running a series of rainfall hyetographs through a simple runoff model. Since synthetic storms are not real rainfall events, but rather an aggregation of data from a number of discrete events, the engineer can then satisfy himself that at least the rainfall input he uses bears some relation to reality (ASCE 1986).

If rainfall and runoff data are to be collected, the statistical analysis referred to above will be invaluable in designing a monitoring program. For instance, if the analysis indicates that precipitation is evenly distributed throughout the year, data collected during any part of the year should be adequate. On the other hand, if precipitation is highly seasonal, then monitoring should be done during the critical season(s).

IV. INFORMATION REQUIRED FOR DESIGN

Depending on the type of storm drainage facilities envisioned and the stage of the design process, some or all of the following information will be required for proper design of drainage facilities for an area. In most cases, the information can be obtained from federal, state, or local governments, or through field inspection or instrument surveys.

A. Topographic Information

(a) City, county, USGS or other topographic mapping.
(b) Aerial photographs.
(c) Vegetation maps.
(d) Soil Maps.
(e) Property surveys and maps.
(f) Property ownership maps.
(g) Field investigations or surveys, to determine the following:
  (1) Drainage basin boundaries to confirm interpretation of maps.
  (2) Drainage basin areas and existing and projected land use characteristics (most communities have land use master plans).
  (3) Typical overland flow paths, swales, channels, and major drainageways.
  (4) Ground and drainageway slopes and lengths.
  (5) Typical channel cross sections.
  (6) Sites potentially suitable for detention storage.
  (7) All relevant drainage and flood control facilities, such as culverts, bridges, drop structures, and utilities crossing channels.
  (8) Properties that have actually sustained flood damage in the past.

The goal of field surveys is to confirm data obtained from maps and to assure that there are no unusual circumstances associated with the study area such as diversions out of the basin, or ponds that mapping does not show.

If adequate mapping is not available, the designer or his client will have to arrange for its preparation. Mapping needed for final design of local drainage systems typically requires a scale of 1 inch = 100 feet to 1 inch = 200 feet, with 1- or 2-foot contour intervals.

B. Survey and Boundary Data

(a) Land boundaries and corners.
(b) Bench marks.
(c) Aerial photographs and ground control.
(d) Existing streets, alleys, railroads, power lines, canals, schools, parks, and other physical features that will influence project feasibility and siting.
(e) Location of utilities.
(f) Existing rights-of-way and easements, along with their characteristics.

(g) Any other potential impediments to drainage system installation, such as community open space, protected habitat, etc. (a "site audit" for hazardous wastes or other contamination is often advisable).

C. Soils and Geologic Data

Excellent sources of soils and geologic data include U.S. Soil Conservation Service (SCS) soils reports, and files of state universities and departments of agriculture.

(a) Infiltration characteristics and permeability of basin soils.
(b) Other standard soil characteristics such as gradation, density, and classification.
(c) Bed and bank samples from drainageways and laboratory evaluations, if applicable.
(d) Soil strength properties for assessments of excavation stability and foundations for drainage structures.
(e) Data required to assess suitability of dam sites if detention ponds are envisioned.
(f) Groundwater elevations on a seasonal basis.
(g) Bedrock locations and characteristics, especially if foundations or trenches are involved.
(h) Other geotechnical characteristics required by either the hydraulic or structural designer.
(i) Local geologic maps or reports that could influence the project.
(j) Any other natural hazards that could affect the drainage system such as landslides or earthquakes.

D. Hydrologic and Hydraulic Data

This is a complex area, and the reader is advised to consult Chapters 5 and 6 for further information. The following list provides general guidance.

(a) Historical streamflow, sewer flow, and precipitation data.
(b) Local rainfall data, including records from local weather stations, intensity/duration/frequency curves, hyetographs, and design storm distributions.
(c) Channel and pipe characteristics including slope, roughness coefficients, vegetation, stability (erosivity), state of maintenance, including amount of debris, etc.
(d) Water "flow line" elevations required for hydraulic grade line analysis. Also sub-basin slopes for analysis of times of concentration and model input.
(e) Existing hydraulic structures including storm drains, inlets, culverts, channels, embankments, bridges, dams, ponds and other similar items.
(f) Existing intentional and inadvertent storage areas and associated flood routing characteristics.
(g) Water quality data.
(h) As much information on large historic floods in the vicinity of the study area as can be derived through library research, interviews with professionals and residents, and other sources.

E. Regulatory Data

One of the key ingredients of a successful design effort is regular, frank interaction between the designer and review authorities. Good communication results in well-conceived designs that benefit, and are well received by, the public (Edwards 1982). Ideas are frequently put forward that otherwise would not have influenced the design, and that help the designer and his client avoid surprises.

It is of particular importance for engineers to become familiar with local drainage goals, objectives, policies, and criteria (Texas Public Works Association 1986). Local codes and standards set the ground rules for designing storm drainage facilities, but they do not relieve the engineer of his responsibility to design a safe system (Wright 1979). The engineer should consult with the department charged with administering local standards and codes to obtain pertinent information. Where these codes and regulations are absent or incomplete, the engineer should advise the regulatory agency of standards needed to protect the public interest. Examples of the sorts of information typically required can be found in Denver Urban Drainage and Flood Control District (1984), King County (1990), ASCE (1976), Florida Department of Environmental Regulation (1988), and U.S. Department of Transportation (1979), and can include:

(a) Zoning ordinances/maps.
(b) Floodplain zoning and requirements (is community participating in National Flood Insurance Program?).
(c) Subdivision regulations.
(d) Building and health codes.
(e) Water and sewer standards.
(f) Erosion, grading, water quality and environmental protection ordinances.
(g) Maps that show designated wetlands, wildlife habitat, receiving stream standards and classifications and other items related to environmental protection (wetlands are an especially important design consideration).
(h) Stormwater management policy and criteria documents, including representative stormwater master plans and final design drawings and specifications.
(i) Comprehensive land use planning reports and other information from local planning departments.
(j) EPA 208 plans (available from local government).
(k) Site specific or adjacent stormwater master plans.
(l) U.S. Soil Conservation Service PL-566 plans.
(m) Flood insurance studies and maps.
(n) Regional flood studies from ungauged basins, normally prepared by U.S. Geological Survey.
(o) Drainage reports and plans which detail specific functions and projected future changes for relevant upstream and downstream facilities or projects, especially if the designer intends to rely on these facilities for the proper functioning of his facilities.
(p) Stormwater NPDES permits.

V. FINANCIAL DATA

Collect information related to existing policies, obligations, or commitments that bear on the financing proposed for drainage facilities (to assure that adequate capital, operation and maintenance funds will be provided). Determine availability of federal, state, and local aid for projects. Determine local construction conditions that may affect project costs through review of bid prices, interviews with contractors, assessment of the state of the local economy, time of the year that the project will be built, etc.

VI. DATA MANAGEMENT

All of the data collected should be organized, reviewed for thoroughness and applicability, and disseminated to the owner, regulatory officials, members of the design team, and others as appropriate. Information should be arranged neatly and systematically to facilitate rapid retrieval.

The microcomputer allows even small engineering firms to manage large amounts of data with relative ease using a variety of commercially-available programs. These programs include both generalized data management software as well as specific data management packages tailored to the needs of engineering organizations. There are, in addition, many sources of computerized data. For instance, USGS daily streamflow records and NOAA daily weather summaries for the U.S. (and similar data for Canada) are available in microcomputer-compatible formats (floppy disks or CD/ROM). Successful stormwater master planning requires thorough and well-organized data, and the maintenance of a computerized data base or, at the very least, detailed project data files, is strongly recommended.

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Chapter 4

DESIGN CONCEPTS AND MASTER PLANNING

I. INTRODUCTION

The conceptual design step is where objectives of the project are delineated in a manner consistent with stormwater control principles, water quality objectives, and local drainage policies. Design concepts provide a road map for preliminary and final design work. During the conceptual stage, the designer formulates the outline for and begins the "drainage master plan study" for the project, a report with accompanying drawings, photographs, maps and calculations, that discusses such subjects as the nature of the problem, hydrology, land use, alternatives evaluated, cost considerations, and environmental impacts. A master plan is normally prepared for both the client and review authorities (Sheaffer et al. 1982).

The total stormwater control system comprises a wide array of physical components, and includes overland flow paths, gullies, channels, streams, detention storage, floodplains, and larger downstream storage/treatment facilities and natural storage areas. The way in which natural and man-made components of the drainage system interrelate at the conceptual design level is the focus of this chapter.

II. PRINCIPLES

Experience has shown that the following general principles apply when planning for and designing urban storm drainage systems (King County 1990; Livingston et al. 1988; Urbonas and Roesner 1991; Schueler 1987; Urban Drainage and Flood Control District 1984)

(a) **Drainage is a Regional Phenomenon That Does Not Respect the Boundaries Between Government Jurisdictions or Between Public and Private**

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Properties. This makes it necessary to formulate programs that include both public and private involvement. Overall coordination and master planning must be provided by the governmental units most directly involved, but drainage planning must be integrated on a regional level if optimum results are to be achieved. The ways in which proposed drainage systems fit existing regional systems must be quantified and discussed in the master plan.

(b) **Storm Drainage Is a Sub-System of the Total Urban Water Resource System**. Stormwater system planning and design must be compatible with comprehensive regional plans and should be coordinated particularly with planning for land use, open space and transportation. Erosion and sediment control, flood control, site grading criteria and regional water quality all closely interrelate with urban stormwater management. The master plan should normally address all of these considerations.

(c) **Every Urban Area has Two Drainage Systems, Whether or Not They Are Actually Planned For and Designed**. One is the **minor** or **primary** system, which is designed to provide public convenience and to accommodate relatively moderate frequent flows. The other is the **major** system, which carries more water and operates when the rate or volume of runoff exceeds the capacity of the minor system. Both systems should be carefully considered.

(d) **Runoff Routing Is a Space Allocation Problem**. The volume of water present at a given point in time in an urban region cannot be compressed or diminished. Channels and storm sewers serve both conveyance and storage functions. If adequate provision is not made for drainage space demands, stormwater runoff will conflict with other land uses, will result in damage, or will impair or even disrupt the functioning of other urban systems.

(e) **Planning and Design of Stormwater Drainage Systems Generally Should Not Be Based on the Premise That Problems Can Be Transferred From One Location to Another**. Urbanization tends to increase downstream peak flow by increasing runoff volumes and by increasing the speed of runoff. Stormwater runoff can be stored in detention reservoirs, which can reduce the downstream drainage capacity required.

(f) **An Urban Drainage Strategy Should Be a Multipurpose, Multimeans Effort**. The many competing demands placed upon space and resources within an urban region argue for a drainage management strategy that meets a number of objectives, including water quality enhancement, groundwater recharge, recreation, wildlife habitat, wetlands creation, protection of landmarks/amenities, control of erosion and sediment deposition, and creation of open spaces.

(g) **Design of the Stormwater Management System Should Consider the Features and Functions of the Natural Drainage System**. Every site contains natural features that may contribute to the management of stormwater under existing conditions. Existing features such as natural drainageways, depressions, wetlands, floodplains, permeable soils, and vegetation provide natural infiltration, help control the velocity of runoff, extend the time of concentration, filter sediments and other pollutants, and recycle nutrients. Each development plan should carefully map and identify the existing natural system. "Natural" engineering techniques can preserve and enhance the natural features and processes

of a site and maximize post-development economic and environmental benefits, particularly in combination with open space and recreational uses. Good designs improve the effectiveness of natural systems, rather than negate, replace or ignore them.

(h) **In New Developments, Stormwater Flow Rates After Development Should Approximate Pre-Development Conditions, and Pollutant Loadings Should be Reduced**. Three interrelated concepts should be considered:

(1) The perviousness of the site should be maintained to the greatest extent possible.

(2) The rate of runoff should be slowed. Preference should be given to stormwater management systems which use practices that maintain vegetative and porous land cover. These systems will promote infiltration, filtering and slowing of the runoff. It should be noted that it may be difficult to restrict post-development **volumes**, and that existing storm water regulations instead require control of **peak flows** to pre-development levels. This may present no problems if the basin has a positive outfall to a stream or river. It can be a problem, however, for a small enclosed basin draining to a lake. Even if retention is provided, the lake might rise because of additional inflow via the shallow water table, and, more importantly, because the increased imperviousness reduces the area available for evapotranspiration. For such basins, the total water budget should be considered, not just peak flows.

(3) Pollution control is best accomplished by implementing a series of measures, which can include source control, minimization of directly connected impervious area (see also Chapter 12), and construction of on-site and regional facilities, to control both runoff and pollution.

(i) **The Stormwater Management System Should Be Designed, Beginning With the Outlet or Point of Outflow From the Project**. The downstream conveyance system should be evaluated to ensure that it has sufficient capacity to accept design discharges without adverse backwater or downstream impacts such as flooding, streambank erosion and sediment deposition.

(j) **The Stormwater Management System Should Receive Regular Maintenance**. Failure to provide proper maintenance reduces both the hydraulic capacity and pollutant removal efficiency of the system. The key to effective maintenance is the clear assignment of responsibilities to an established agency and a regular schedule of inspections to determine maintenance needs and to insure that required maintenance is done. Demonstrated past local maintenance performance should be the basis for the selection of specific design criteria.

III. DRAINAGE SYSTEMS

A. Natural Channels

Most of the time the natural system (hills, valleys, lakes, stream channels, floodplains, wetlands and coastal plains, etc.) appears to be

in equilibrium. This "equilibrium" is relative, however, since changes due to erosion and deposition occur continually (Urban Land Institute 1975, 1978).

The size, shape and slope of a stream channel are functions of interrelated variables such as soil characteristics, lithology, width, depth, velocity, slope, sediment load, sizes of sediment and debris, hydraulic roughness and discharge. The stream channel accommodates itself to whatever discharge it receives. Increasing the flow alters the overall size, but the shape of the channel margins tends to remain constant. The cross-sections of most streams tend to be generally trapezoidal in straight reaches, but are asymmetric at curves or bends. They tend to become more rectangular as the stream becomes larger downstream, since width increases faster downstream than depth. Depth usually increases faster downstream than does velocity. Width usually increases in a more consistent manner than any other factor, roughly as the square root of the discharge; and mean velocity tends to decrease slightly downstream in most rivers (U.S. Bureau of Reclamation 1977).

In a sediment-laden stream, flow resistance is altered by the size of the bed particles, the form or configuration assumed by the particles on the channel bed, and the damping of turbulence by the sediment load. Channel width and depth increase, stream gradients generally flatten, and river bed particle sizes usually diminish as one moves downstream. A constant or gradually steepening stream gradient therefore would be associated with increasing bed particle sizes. A stable stream is one that reaches a slope that provides just the velocity required to transport the sediment load supplied from the drainage basin. Any change in the controlling factors will cause displacement of the equilibrium in a direction that will tend to absorb the effect of the change (Kolenkow et al. 1974; Linsley et al. 1982).

B. Effects of Urbanization

Urbanization disrupts the natural equilibrium of streams. Construction site erosion can result in order of magnitude increases in local sediment loads. The increased imperviousness and hydraulic efficiency of the urban flow paths can generate local (and sometimes quite large) increases in peak runoff rates. The effect on the stream increases with the percentage of the watershed that has undergone urbanization. Increases in water and sediment loads can be minimized by properly designing components of the urban drainage systems such as detention basins and on-site erosion control measures during construction phases. If such increases are not minimized, the natural streams will enlarge their channels to accommodate the increased loads by scouring their banks and beds, thus generating additional sediment loads (Urban Land Institute 1975, 1978; U.S. Soil Conservation Service 1973).

IV. BASIC CONCEPTUAL ELEMENTS

In some areas, urban stormwater management is being perceived and treated, at least in part, as a subsystem of a broader urban water resources management system. (This ideal, unfortunately, is not often seen, however, due to the "balkanized" (McPherson 1978) nature of the various components—water supply, waste disposal, flood control, urban drainage.) The advantage of overall urban water resources management is that urban drainage can be interrelated with such functions as groundwater management, water supply, waste disposal, slope stability control, aesthetic and recreational opportunity control, and others (King County, Washington 1990; Livingston et al. 1988; State of Delaware 1990).

As the capacity of the minor system is exceeded, streets or other surface channels (the major system) begin to carry excess flow. This condition does not constitute failure of the storm drainage system. This concept has implications for design and for analysis of total system costs, since the majority of costs are for small-diameter pipe and appurtenances in local neighborhoods. For the smaller, more frequent runoff events, the longer the runoff can be kept on the surface, the shorter and less costly the storm sewer system will be.

Denver, Colorado—A plaza that adjoins a major river. The lowest level of the plaza is inundated during the 2-year runoff event, and the 100-year event floods the entire plaza.

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For the larger, less frequent storms, the designer should determine, at least in a general sense, the flow pathways, and related depths and velocities, of the major system. **This does not mean that a detailed analysis of the major system need be made in all cases, nor that existing systems, whose performance may be well understood, need always to be subject to this sort of analysis.** It is, rather, directed primarily at new developments, or to significant modifications to existing systems. The objective is to keep water out of buildings, and to assure that flow depths and velocities will not constitute a hazard (or at least to be able to define the hazard) to public convenience and safety.

Designs may combine portions of both the major and minor systems into a single system. One set of flow paths, both natural and man-made, can convey the runoff from both small and large storms without causing damage. It requires imagination, initiative, and ingenuity on the part of designers, developers, and local governments to incorporate such flow paths into the city in a way that is compatible with the urban setting. The result can be an overall system that costs less, is aesthetically pleasing, and is free from damage from all but catastrophic flood events (Urban Drainage and Flood Control District 1984).

V. PREVENTIVE AND CORRECTIVE ACTIONS

In existing urbanized areas, the designer will frequently find it necessary to develop a strategy based upon both preventive and corrective measures. Structural corrective actions affect and control the storm runoff and floodwaters directly, and can encompass such things as inlets, storm sewers, interceptor lines, channelized stream sections and reservoirs. There are also non-structural corrective actions which limit activities in the path of neighborhood storm runoff or in river floodplains. They include floodproofing and land use adjustments (Sheaffer et al. 1982).

A. Preventive Actions

Preventive actions available for reducing storm runoff and flood losses includes (Flood Insurance Administration 1981):

(a) Control of flood-prone land uses.
(b) Floodplain regulation.
(c) Flood-prone land acquisition.
(d) Subdivision regulations.
(e) Building code provisions.
(f) Control of water and sewer extensions.
(g) Flood-prone area information and education.
(h) Storm and flood forecasts and emergency measures.
(i) Measures to reduce the runoff rate.
(j) Measures to reduce erosion.
(k) Floodproofing (both preventive and corrective).

This slope (in the Pacific Northwest) should never have been developed. All of the homes shown were destroyed by a landslide which occurred shortly after this photograph was taken.

The designers of urban drainage systems must recognize that flood-prone lands and floodplains exist not only along rivers and streams, but also in headwater residential and commercial neighborhoods where storm drains are lacking or inadequate, where building floor elevations are too low, and where curbs and gutters are subject to overtopping.

By controlling the amount and type of economic and social growth in the floodplain, flood losses may be reduced and net benefits from suitable floodplain use increased (FEMA 1981a, 1981b). For instance, it may be found that it is more economical to turn a block of homes into a small park than to solve the storm drainage problem for an isolated area. Although this approach often requires a long period to realize its full effect, the community benefits can be significant. They include reduction in the exposure to risk, reduction in public costs for relief and rehabilitation, and decreased dependence on protective works. Public acquisition of properties at risk, with rent-back provisions that include flood insurance coverage requirements, can be used for long-term flood loss reduction at minimum public cost (FEMA 1981a, 1981b).

B. Delineation of Floodplains

The delineation of flood-prone areas is the first step in floodplain regulation and local drainage design. The runoff is sometimes calculated

based upon the projected future development of the basins in accordance with long-term area-wide plans, and assuming storm sewers, surface drainage, floodplains, and watercourses in their existing condition. Adjacent properties that may discharge overland flow onto the study area should also be evaluated.

Acquisition of floodplain land may be a cost-effective means of avoiding unacceptable property damage. In addition, other benefits may be realized, such as the preservation of floodwater storage capacity and the provision of recreational open space.

C. Corrective Actions

Existing drainage, flood hazard, and water quality problems may require corrective actions, even under a basically non-structural management plan. These corrective actions fall into categories as follows:

(a) Construction of storm sewers, stormwater storage, and water quality best management practices (BMPs).
(b) Land use adjustments.
(c) Channelization to enlarge streams.
(d) Enlarging bridge and culvert openings.
(e) Modifying bridge and culvert approach, entrance, and discharge transitions.
(f) Designation of nonconforming uses.
(g) Floodproofing of buildings.

Land use adjustments include rezoning, relocation of structures, programmed removal of incompatible structures, and purchase of floodplain properties (which may be leased back for temporary use). Urban renewal projects may be used to expedite adjustments in land use.

Rather than constructing new systems, it may sometimes be advantageous to retrofit existing systems to obtain increased capacity or to provide pollution control. Innovative retrofitting practices are continually being developed and improved, (Livingston 1986; Pisano 1990; Torno 1990), and the reader is urged to consult the current references.

It should be noted that, compared to construction in new areas, retrofitting can be expensive, principally because of such factors as land costs and disruption of traffic and/or utilities.

VI. STRUCTURAL COMPONENTS OF DRAINAGE SYSTEMS

The first sections of this chapter have emphasized the need to analyze drainage requirements in the context of the entire urban system, to relate the effects of local drainage solutions to their cumulative impact upon the larger drainage system, and to consider a wide range of non-structural and land-use measures in the design. Equal importance must

be placed on the structural components of the drainage system. The following list of structural components and design considerations is not intended to be comprehensive, nor is the entire design process from start to finish explicitly summarized.

A. Major Drainageways

(a) Where are they now and where should they be after development?
(b) How far up into the basin should they extend?
(c) How will they behave in various floods?
(d) What are important geomorphologic characteristics?
(e) What channel improvements exist upstream and downstream?
(f) What does the floodplain look like presently, and can and/or should it be narrowed?
(g) How much land should be allocated for flood conveyance, and is this adequate? Does it fulfil community objectives?
(h) Are improvements required?
(i) Can/should multi-purpose uses for the channel be encouraged?
(j) Concrete-lined channels
  (1) Is there a limited right-of-way that constrains channel width?
  (2) Will flow be subcritical or supercritical?
(k) Grass-lined channels
  (1) Will a "soft," natural approach to channels fit better with the area?
  (2) Can velocities be adequately limited with drops?
  (3) Is there enough right-of-way for 4:1 or flatter side slopes?
(l) Box culverts
  (1) Box culverts can be used at streets.
  (2) Can internal pressure be controlled in long box culverts?
  (3) The capacity will lower if the culvert reaches full flow.
(m) Large pipes
  (1) Is an underground conduit preferable to an open channel?
  (2) Would an emergency open channel still be needed?
(n) Riprapped channels
  (1) Will a riprapped channel be aesthetically acceptable?
  (2) Stone riprap often is vandalized.
  (3) If wire gabions are used for drops, will the wire protective coating be eroded or corroded causing failure?

B. Streets

(a) What depth and velocity of flow is permitted in the gutters for the primary design runoff event? What depth for the major system runoff? Can velocities be maintained within safe limits?
(b) Can a minimum of 0.5% street slope be maintained? (note that there are existing developed urban areas in which 0.1 percent would be a luxury).
(c) How will flow across street crowns and intersections be handled?
(d) How much depth of flow over the street is appropriate for the major storm runoff, considering local conditions and classes of streets (local, collector, or arterial)?

(e) How much will future pavement overlays reduce the curb and gutter conveyance capacity?

C. Storm Sewers

(a) Will storm sewers be necessary?
(b) Will the storm sewer be designed for pressure flow or open channel flow?
(c) Are trash racks necessary for safety or would another safety feature serve just as well considering the problem of trash rack plugging?
(d) Cleaning of trash racks should be a design consideration, with adequate access for equipment during a major storm runoff.

D. Storm Inlets

(a) Where are curb opening inlets optimal? Where should grated inlets, sump inlets, combination inlets be used?
(b) How much flow will bypass inlets under various street grades?
(c) How much inlet capacity is required for full storm sewer capacity utilization?
(d) Should inlet capacity be limited to avoid over-charging the storm sewers in one location to the detriment of another area during the major storm runoff?
(e) The storm sewer design for the minor system design runoff should be compatible with the major runoff event surface flow.

E. Intersections

(a) Should street intersections be kept free from surface flows for the minor system design runoff? Should cross pans be used? What is the effect on traffic?
(b) How should tee intersections be handled at the foot of a steeply sloping street?

F. Flow Control Devices

(a) Energy dissipators are useful when changing from concrete-lined channels to grass-lined channels if velocities would otherwise be excessive, or anywhere else that hydraulic energy should be reduced.
(b) Hydraulic drops are necessary to keep channel velocities within design limits when the channel slope is excessive.
(c) Acceleration chutes are used when making a transition from slow-flow channels to higher velocity channels.
(d) Bends in an open channel need to be hydraulically analyzed for head loss, overtopping of the outside banks, and effectiveness.

G. Trash Racks (Safety Racks)

(a) Is a trash/safety rack essential?
(b) Will trash racks cause a system failure if they become plugged?
(c) What maximum and minimum velocities should be selected?

(d) Should detention/retention storage be used to reduce the required size of storm sewers?
(e) What construction problems are likely to be encountered and how will they affect costs?
(f) Where will the system discharge? Water quality may be an issue worthy of attention.
(g) Will downspouts/foundation drains be connected to the storm sewers? This has been common practice, primarily because the sewers provide a convenient means of disposal, but this conflicts with the notion of minimizing the impervious area directly connected to the storm sewers, reduces sewer capacity, and, in the case of foundation drains, can increase the risks of basement flooding. This issue should be carefully looked at when-retrofitting existing systems.

H. Detention Facilities

(a) Is detention required? If yes, what are the appropriate regulations? Is capacity dictated by local ordinance or policy?
(b) What are the design discharge frequencies?
(c) What kinds of detention are appropriate? Is on-site ponding necessary, or should regional (large) ponds be relied upon?
(d) How will contemplated detention facilities address erosion/sediment control, runoff quality enhancement, creation of attractive and safe park areas, groundwater recharge, and other multi-use considerations?
(e) How will detention fit into the regional drainage system? For example, could on-site detention actually aggravate, rather than reduce, downstream peaks?
(f) If a dam is required, then standard dam design considerations and studies are necessary including geotechnical evaluations, adherence to state regulations for design flood spillway capacity, freeboard, hazard classification, etc.

I. Water Quality Mitigation Measures (Other Than Detention)

(a) What local requirements prevail (how much of which pollutants must be removed, and how frequently).
(b) What mitigation measures can be adopted, and how can they be optimized.

J. Other Special Structures

(a) How many special structures (outlet and inlet protection, flow splitters, multiple channel lining types, diversion boxes, etc.) will be necessary?
(b) What design considerations are associated with them?
(c) Can some be eliminated or modified?

It is characteristic of drainage system design problems (and water engineering in general) that the design considerations mentioned above cannot be reduced to simple rules. This is why imagination, experience and mature judgment play equally important roles in the conceptual design phase of successful urban drainage projects (see Chapter 9 for further details).

VII. RISK ANALYSIS

An essential step in the design of urban drainage and flood control systems is the selection of the recurrence frequencies (probabilities) of the runoff events for which the major and minor systems are to be designed, which in turn will determine the sizing of the various components of the systems. A clear understanding of the risks and costs associated with alternative drainage designs leads to better drainage systems and to wiser public and private investment. The process by which this understanding is achieved is risk analysis, the elements of which are described briefly herein. The additional cost of risk analysis (engineering analysis, regulator education, and negotiation) may restrict its application to larger projects.

There are no hard and fast rules regarding recurrence frequencies for design. The engineer must ascertain what local policy is regarding design return frequencies, and then ask if such standards are appropriate for the particular setting. Common sense and judgment on the part of the designer and local authorities should supersede uniform but arbitrary standards. Local regulators may accept (or at least carefully consider) proposed deviations based on principles of risk assessment.

A. Definitions

Risk is the expression of potential adverse consequences measured in terms of inconvenience, damage, safety, or even professional liability or political retribution. Risk analysis is the quantification of exposure, vulnerability and probability. Risk analysis involves the evaluation of alternative means to reduce risk and, finally, the determination of acceptable levels of risk (Wiggins 1978).

In a risk analysis context the design runoff event is the event the drainage system must handle without permitting an unwanted consequence (unacceptable risk). It implies that uncertainty has been defined (the return period or the probability of the event), acceptable risk has been determined (no event smaller than the design event will exceed the capacity of the primary drainage system), and the vulnerability of the finite number of exposed improvements has been quantified and found consistent with the acceptable risk.

Because some of the criteria related to drainage design have come to be accepted as principles, there is confusion about their meaning and application. For example, the "100-year runoff event" is the event that has a probability of occurrence of 0.01 in any given year. It is often taken to mean that the event will occur only once in one hundred years, which although true on the average, may not be true for a particular 100-year period. Furthermore, within the context of risk analysis, the unwanted consequence is not the runoff but the damage which results. It is the uncertainty of this consequence which is of concern, and not the occurrence of the event.

The extent to which risk analysis is employed in the design and selection of a drainage system depends on the nature and extent of the unwanted consequences. For simple situations, mere recognition of risks, uncertainty and possible adverse consequences may be sufficient to permit informed judgments. When public health and safety are at stake, a more rigorous risk analysis should be undertaken. In any event, however, the elements of risk analysis should be systematically considered in formulating design parameters and guiding the decision-making process.

B. Methodology

The determination of risk-causing factors and unwanted consequences requires an assessment of the probability of occurrence and value of potential damage. Four determinations must be made. First, the critical events (i.e., the design events) must be defined and their magnitudes estimated. For drainage design, these events should be those that produce the unwanted consequences. Only the events that would cause damage need to be evaluated, and a probability of occurrence determined for each.

Next, the vulnerability of the exposure (the property that can be damaged) should be determined. The consequence, the third element, might range from public safety through inconvenience to severe damage. Finally, a value (such as dollars or time delays), for all exposed properties, is associated with each of the consequences.

For a given design, the probability of flooding, expected damage and system cost can be assessed. The resulting estimated probability, damage and cost give one point on the risk analysis curve. Similar analyses for alternative designs provide additional points. Information provided by the curve then allows assessment of risk avoidance or acceptance.

Alternative strategies should be pursued to the extent that their marginal costs of implementation are equal. This is to say that each additional dollar spent on avoidance should lower the risk an equivalent amount. The selected design should be the one that balances avoidance and acceptance at the point of acceptable cost and risk.

VIII. DESIGN ECONOMICS

While elements of design are important, information on the costs associated with stormwater management is also necessary to judge the feasibility of a program. Cost information is important to planning agencies and policy makers because the information can be used in setting user fees and comparing alternative basin-wide drainage strategies. Cost information is important to land developers because the information is basic to the determination of the feasibility of development.

Costs for stormwater control are not simple to quantify because of the uncertainties involved. The simplest estimation method consists of a table of mean values, one for each type of development. Such tables are usually applicable only for the jurisdiction for which they were developed. The mean value is the cost per acre of development and is limited to certain cost categories such as construction costs. Costs for such items as engineering, land, and annual operation and maintenance would be estimated separately, possibly by using either mean cost per acre values, or proportionality constants multiplied by the cost of construction.

There are empirical equations for predicting costs associated with other aspects of stormwater control such as erosion and sediment control. Mean cost estimates can be used for such erosion and sediment control methods as small sediment basins, interceptor berms, chemical control, and seeding with fertilizer and mulch (Schueler 1987).

Recognizing that many drainage systems include some form of stormwater runoff control, such as detention, statistical relationships for predicting the construction costs (per unit volume) of detention storage are useful. Similar methods are useful for estimating other costs including land costs, planning, design and supervision costs. Further discussion of economic evaluation is provided in Chapter 8.

IX. DRAINAGE MASTER PLANNING

Master planning is one of the most widely used and frequently misunderstood terms in drainage practice. There are few published definitions, but a master plan typically addresses such subjects as characterization of site development, grading plan, peak rates of runoff and volumes for various return frequencies, locations, criteria and sizes of detention ponds and conveyances, measures to enhance runoff quality, pertinent regulations and how the plan addresses them, and consistency with secondary objectives such as public recreation, aesthetics, protection of public safety, and groundwater recharge.

In its simplest form, a master plan may only identify the essential elements, alignments, and functions of a drainage system. Even at this conceptual level, the master plan should be based upon estimates of peak and total discharges for some selected runoff recurrence interval(s). These recurrence intervals, in turn, should be selected based on local standards and risk assessment, as discussed earlier in this chapter.

The next level of master planning should establish specific criteria consistent with acceptable risk, including design discharges and water surface profiles and elevations. Head losses at waterway crossings and other constructions or obstructions should be recognized in development of the water surface profiles. This level of master planning defines the ultimate drainage system components desired and provides information for their preliminary design and cost estimation. More impor-

tantly, it provides basic information useful to assess the feasibility and practicality of contemplated system components and permits local designs to be undertaken with reasonable assurance that they will be compatible with the ultimate overall system.

Where some components of a drainage system must be complete and in operation before other components may be wisely provided, the drainage master plan should identify essential component completion priorities.

A community cannot wisely allow development in a drainage basin, particularly when such development occurs over a long period of time and is not uniform throughout the basin, without first establishing all controlling drainage design parameters through drainage master planning. Failure to do so invites significant future flooding and drainage construction problems (Sheaffer et al. 1982).

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Chapter 5

HYDROLOGY AND INTRODUCTION TO WATER QUALITY

I. INTRODUCTION

A. Effect Of Urbanization On Streamflows

Figure 5.1 (Schueler 1987) depicts typical changes in watershed hydrology that can be expected as a result of urbanization. Under undeveloped conditions, losses (abstractions) such as evapotranspiration, canopy interception and soil infiltration tend to be large. Under developed conditions, the increase in the amount of impervious surface area (streets, roofs, parking lots, driveways, and sidewalks) in the watershed increase surface runoff. Wet-weather stream discharges generally increase, while dry-weather discharges may either decrease or increase. Pavements, gutters, and storm sewers convey runoff more rapidly than do natural surfaces. Development-related straightening, cleaning, and lining of natural channels increase flow velocities.

The typical impact of urbanization on the runoff hydrograph is shown qualitatively in Figure 5.1b. The post-development hydrograph differs from the pre-development hydrograph in three important ways: (1) the total runoff volume is greater, (2) the runoff occurs more rapidly, and (3) the peak discharge is greater. The increase in runoff volume results from the decrease in infiltration and depression storage. The shortened time base results from the greater flow velocities in the drainage system. The increase in peak discharge is the inevitable consequence of a larger runoff volume occurring over a shorter time. This increase in peak discharge for any storm means a related high discharge occurs more frequently. **Urbanization has a greater impact on frequent events than on rare events.** Because urbanization tends to reduce both shallow subsurface flow and groundwater recharge, dry-weather stream discharges may decline. However, in arid regions, the intensive lawn irrigation that often accompanies urbanization may increase dry-weather flows.

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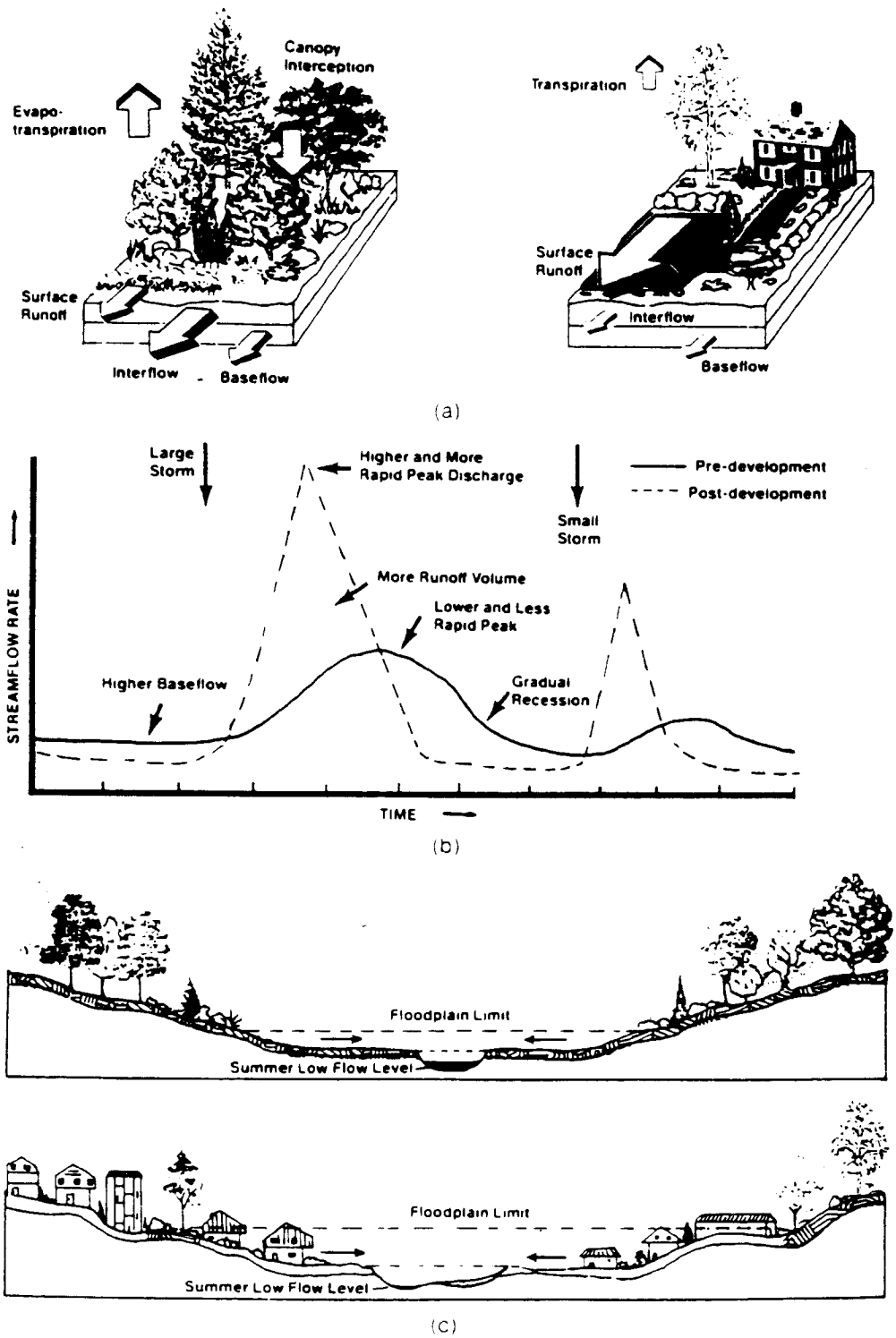

Figure 5.1—Changes in watershed hydrology as a result of urbanization: (a) water balance, (b) streamflow, and (c) response of stream geometry (Schueler, 1987).

This chapter describes some commonly used methods for computing peak discharges and runoff hydrographs for individual rainfall events. Modelling the long-term hydrologic and water quality response of urban watersheds to continuous climatological inputs is addressed in Chapter 7.

B. Quality Of Urban Runoff

Storm runoff may contain significant concentrations of sediment and other substances classified as pollutants. Classes of pollutants typically monitored in stormwater quality studies include suspended solids, heavy metals, nutrients, organics, oxygen-demanding substances, and bacteria. The urban transportation system can be a major source of pollutants. Exhaust emissions contribute heavy metals and hydrocarbons. Some snow and ice control measures contribute sand and salts. Construction activity can be a major source of soil sediments. Other sources of pollutants include particulate emissions from heating systems and industrial processes, fertilizers and pesticides washed off lawns, and lawn clippings, used crankcase oil, washoff from commercial establishments such as filling stations, and solvents dumped in the drainage system.

The effects of urban runoff on receiving waters depend on the hydrology, chemistry, and beneficial uses of the receiving body, as well as the quantity and quality of the discharges. This manual will describe general classes of receiving water impacts that can occur. Because receiving-water impacts are highly site-specific, local evaluations, such as instream biosurveys, sampling of sediment quality, and measurements of discharge quantity and quality, can provide the best information on those impacts. Receiving water impacts are caused by a combination of physical and chemical effects. More frequent occurrences of high discharges may cause or intensify channel erosion problems, disrupting the riparian habitat both where the erosion occurs and where the additional sediment is deposited downstream. Development-related changes in water quality and dry-weather flows may also alter the riparian habitat.

Later sections of this chapter provide a brief introduction to sources, concentrations, and loadings of pollutants in urban stormwater. Chapters 10 through 12 address receiving water impacts and mitigation measures.

II. QUANTITY OF STORMWATER

A. Overview

Whenever the rainfall exceeds the interception by vegetation and infiltration into the soil, water accumulates on the catchment surface. This water flows overland for some distance, filling surface depressions along the way, before concentrating in small channels such as gutters

and swales. These small channels convey the surface runoff to underground storm sewers or larger natural or man-made drainage channels. These storm sewers and larger channels comprise the stormwater transport system. The hydrologic methods described in this chapter are useful for computing runoff hydrographs and peak discharges on small urban catchments. These catchment flows comprise the inflows to the stormwater transport system. The hydraulics of flow in the stormwater transport system are covered in Chapter 6.

To compute and interpret stormwater flows, the engineer must perform the following basic steps:

(a) Define the locations at which flows are required and the corresponding tributary areas.
(b) Determine the present and projected land uses and other physical characteristics for each catchment.
(c) Specify the return periods for which runoff hydrographs or peak discharges are to be computed.
(d) Select appropriate hydrologic procedures for computing runoff hydrographs or peak discharges.
(e) Construct a design storm hyetograph based on local rainfall characteristics.
(f) Compute rainfall abstractions (losses) and thereby rainfall excess.
(g) Compute runoff hydrographs or peak discharges.
(h) Assess the reasonableness of the computed flows.
(i) Evaluate the significance of the computed flows.
(j) Evaluate the significance of any data available on historic floods.

B. Design Points, Catchments, and Return Periods

The first step in surface runoff analysis is identifying the points at which flows must be determined, and their corresponding catchments. For computational purposes, it is often advantageous to divide a catchment into sub-catchments with more or less homogeneous physical characteristics. The outlet of each sub-catchment is termed a design point. Pipes and channels have design points wherever significant quantities of additional flow are introduced. Sub-catchments are linked together computationally by means of a hydraulic or hydrologic routing procedure. Catchment discretization is generally based on drainage patterns, surface slopes, and land use patterns. Sub-catchments may vary in size from a few acres to tens of acres. Areas of catchments and sub-catchments are usually determined from topographic maps, maps showing existing underground storm drains, and grading plans.

Existing and projected drainage patterns, land uses, and physical characteristics for each sub-catchment should be estimated carefully. Current conditions can be determined from field inspection, topographic maps, soils maps, drainage-system maps, aerial photographs, and other sources. The engineer should strive to develop a thorough understanding of actual drainage conditions in the catchment. Detailed inspection helps identify such features as depressions that do not con-

tribute runoff, grading that disrupts natural flow paths, and underground storm drains that cut across surface drainage divides. Most communities have long-range master plans from which future conditions can be estimated, and most regulatory agencies require that stormwater flows be computed on the basis of maximum projected development in the catchment. This helps assure the long-term adequacy of the improvements (it should be noted here that master plans—even "ultimate" plans—can change, often without adequate consideration of the hydrologic consequences).

A key physical characteristic of an urban catchment is the amount of impervious surface area. Impervious surfaces typically comprise between 10–30% of the total surface area in low-density residential districts, between 30–60% in high-density residential districts, and between 80–100% in central business districts. The impervious area within a catchment should be further divided into two categories: directly connected impervious area and non-connected impervious area. Directly connected impervious surfaces drain directly to storm sewers or drainage channels. Examples of impervious surfaces that are usually directly connected include parking lots, streets with curbs and gutters, and roofs with drains leading directly to storm sewers. Non-connected impervious surfaces are impervious surfaces that drain onto pervious surfaces, such as roofs that drain onto lawns.

Peak discharges on small urban catchments may be determined largely by the runoff from directly connected impervious surfaces, particularly for the more frequent storms. It is therefore important that the directly connected impervious area in a catchment be estimated with reasonable accuracy. Driveways, sidewalks, playgrounds, roofs, and streets without curbs and gutters might or might not be directly connected, depending on site-specific conditions. When analyzing an existing drainage system, impervious surfaces should be inspected in the field to determine whether or not they are directly connected.

Selection of design return periods for the minor (primary) and major drainage systems is discussed in Chapter 8. Regardless of the design basis, it is recommended that performance of the drainage system be examined for a range of return periods. A comparison of results for the various return periods may indicate the need for a different design basis.

C. Methods For Computing Stormwater Flows

There are two basic approaches to computing stormwater flows. The first approach relates runoff to rainfall through a proportionality factor and yields only a peak discharge. This approach, termed the rational method, has been used by engineers since the nineteenth century. The second approach starts with a rainfall hyetograph, accounts for abstractions and temporary detention in transit, and yields a discharge hydrograph. Most modern methods use the hydrograph approach.

Traditionally, design discharges for street inlets and storm sewers have been computed using the rational method, although hydrograph methods also can be used for these purposes. The primary attraction of the rational method has been its simplicity. However, now that computerized procedures for hydrograph generation are readily available to all engineers, computational simplicity no longer need be the primary consideration. Experience has shown that the rational method can provide satisfactory estimates of peak discharge on most small catchments. For larger catchments, storage and timing effects can become significant, and therefore one of the hydrograph methods should be used. The rational method cannot be used to design a detention storage facility, because a complete hydrograph is needed. Various methods have been devised to form pseudo-hydrographs based on the rational formula, but their reliability is uncertain.

Sections D–F present some concepts and methods that are useful for developing design hydrographs for storm drainage facilities. The rational method is covered in Section G. More information on these topics can be found in the references provided, and in textbooks on engineering hydrology (e.g. Chow, Maidment, and Mays 1988; Viessman, Lewis, and Knapp 1989; McCuen 1989; Ponce 1989; Linsley, Kohler, and Paulhus 1982; and Hjelmfelt and Cassidy 1975).

D. Design Rainfall

The specification of a rainfall event, sometimes called a "design storm," as a design criteria is widely used in engineering practice. Despite this widespread use, however, the subject of design storms is controversial. Much of the controversy stems from the lack of realistic and accurate definitions of design storms, and confused thinking about their application. The main criticisms of design storms arise from the practice of assigning a particular frequency to a design storm, neglect of antecedent catchment conditions, and design on the basis of the return frequency of rainfall rather than runoff.

The use of a design storm requires minimal resources, and this, as well as the lack, in the past, of well-defined and inexpensive alternatives, has contributed to the popularity of this approach. Although the design storm must reflect required levels of protection, the local climate, and catchment conditions, it need not be scientifically rigorous. It is probably more important to define the storm and the range of its applicability fairly precisely to ensure safe, economical, and standardized usage.

Two types of design storms are recognized—synthetic and actual (historic) design storms. The former are derived by synthesis and generalization of a large number of actual storms. The latter are events that have occurred in the past, and which may be well documented in terms of their impacts on the drainage system. Most of the following discussion concentrates on synthetic storms.

In catchments with low imperviousness, the pervious segments will control the generation of runoff peaks, and neglect of antecedent conditions may no longer be acceptable. As catchment area increases, there may be more opportunities to incorporate runoff storage into the design, and spatial effects may become more significant. The application of conventional design storms to the design of drainage systems incorporating storage should be avoided, though many engineers use them for this purpose due to the perceived lack of practical alternatives (see also McPherson (1969, 1978) and Adams and Howard (1986) for a comprehensive discussion of limitations of the design storm approach).

1. *Synthetic Design Storms*

Synthetic design storms (which in fact bear little or no resemblance to real rainfall at the site in question) are derived from synthesis and generalization of point-rainfall data for a large number of actual storms. These design storms are normally defined by their duration, total rainfall depth, temporal pattern, spatial characteristics (average spatial distribution, storm movement, and spatial development and decay) and some measure of antecedent rainfall. The total rainfall depth is normally selected so that the depth-duration combination has some specified return period. It should be noted that any frequency associated with the "storm" (rainfall) is likely to be different from the frequency associated with peak runoff flow, runoff volume, or pollutant loading which results from that rainfall.

Design Storm Return Period Ideally, the return period should be selected on the basis of economic efficiency. In practice, however, economic efficiency is typically replaced by the concept of level of protection. The selection of this level of protection (or return period), which actually refers to the exceedance probability of the design storm, rather than the probability of failure of the drainage system, is largely based on local experience. Typical return periods used in the United States and Canada, are given in Table 5.1, although longer return periods are sometimes used.

Storm Duration Design storm duration is an important parameter that defines the rainfall depth or intensity for a given frequency, and therefore affects the resulting runoff peak. In selecting a duration, one should consider both the hydrologic time-response characteristics of the catchment and the typical durations of intense storms in the region. The design storm duration that produces the maximum runoff peak depends on the catchment time constant, traditionally defined as the time of concentration. The time of concentration is commonly expressed as the travel time (properly the **wave** travel time—see Section G. 4) from the most remote point in the catchment to the point under design. Such a definition ignores the relative runoff-producing capabilities of pervious and impervious areas, and possible variations in rainfall intensity.

TABLE 5.1: Typical Design Storm Frequencies

Land Use (1)	Design Storm Return Period (Frequency) (2)
Minor Drainage Systems	
Residential	2– 5 years
High value general commercial area	2–10 years
Airports (terminals, roads, aprons)	2–10 years
High value downtown business areas	5–10 years
Major Drainage System Elements	up to 100 years

Nevertheless, the current practice is to select the design storm duration as one equal to or longer than the time of concentration for the catchment (or some minimum value when the time of concentration is short). Intense rainfalls of short durations (30 minutes or less) usually occur within longer-duration storms rather than as isolated events. It is common practice (Packman and Kidd 1980) to compute discharges for several design storms with different durations, and then base the design on the storm which produces the maximum discharge. Note that the storm durations discussed above may not be suitable for storage design.

2. *Rainfall Depth*

The total storm rainfall depth at a point, for a given rainfall duration and frequency, is given by the local climate. Rainfall depths for various durations and frequencies are published in maps of precipitation and in the reports of governmental agencies that collect precipitation data. Rainfall depths can be further processed and converted into rainfall intensities (intensity = depth/duration), which are then presented in rainfall intensity-duration-frequency (IDF) curves. Such curves are particularly useful in storm drainage design because many computational procedures require rainfall input in the form of intensities.

Although some municipalities collect and process their own rainfall data, the drainage design is more often based on rainfall data compiled and processed by national agencies. In the United States, the best source of rainfall data are the data banks of the National Climatic Data Center, and reports (rainfall frequency atlases) of the National Weather Service. Besides rainfall data, it is possible to obtain maps containing isolines of constant rainfall depth for specific durations and return periods (U.S. Weather Bureau 1958). Technical Paper 40 (Hershfield 1961) contains maps of the entire United States showing rainfall depths for durations from 30 minutes–24 hrs and return periods from 1–100 years. More-detailed maps for eleven western states are found in NOAA Atlas

2 (Miller et al. 1973). This atlas, issued in eleven volumes, contains maps for durations of 6 and 24 hours and return periods from 2–100 years. For the eastern and central U.S., a report by Frederick et al. (1977) provides maps for durations of 5, 15, and 60 minutes and return periods of 2 and 100 years. All three of these reports provide procedures for interpolating, and in some cases extrapolating, to durations and return periods not included in the reports.

Rainfall data from these reports can be abstracted, converted into intensities, and presented in the form of IDF curves. Similarly, the total storm rainfall depth can be determined from such maps for a chosen storm duration and return period.

It should be noted here that, since the last century, underestimation of rainfall (due to wind effects upon rain gauges) has been recognized (Jones 1990). Despite this, published rainfall intensity/duration/frequency data have been developed from these measurements, without adjustments for wind effects. This can be a particular problem in areas where convective thunderstorms (with attendant high winds) account for rainfalls of the highest intensity.

The rainfall intensity-duration curves for individual frequencies also are sometimes approximated by mathematical expressions in one of the following forms:

$$i = \frac{a}{(t_d + c)^b} \tag{5-1}$$

or

$$i = \frac{a}{t_d^b + c} \tag{5-2}$$

in which i is the average intensity for duration t_d, and a, b, and c are fitting constants. The values of the constants vary with location and return period. One should not apply an equation of this type outside the range of the data to which it was fitted. Figure 5.2 shows a family of curves of this type.

Recognizing that the precipitation data in maps were subject to some interpolation and smoothing, it may be justifiable to develop IDF curves directly from local rain-gage records if these records are sufficiently long and reliable. An example of this type of analysis for a single rain gage is presented by Chow, Maidment, and Mays (1988). Wenzel (1982) also describes procedures for performing such analyses.

3. *Temporal Distribution*

The temporal distribution of rainfall within the design storm is an important factor that affects the runoff volume, and the magnitude and

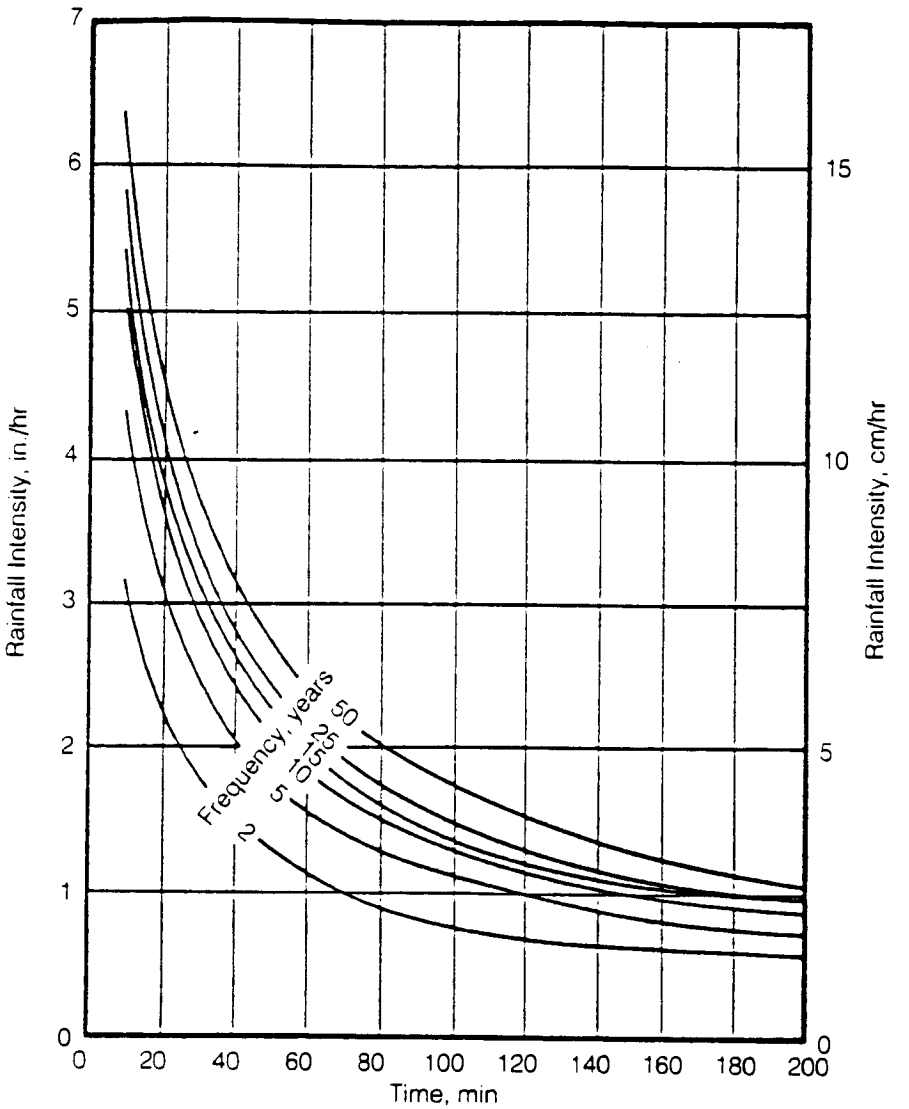

Figure 5.2—Intensity-duration rainfall curves, Boston, Massachusetts.

timing of the peak discharge. Realistic estimates of temporal distributions are best obtained by analysis of local rainfall data from recording gauge networks. Such an analysis may have to be done for several widely varying storm durations to cover various types of storms and to produce distributions for various design problems. Where such analyses cannot be justified, the designer must adopt one of the existing distributions. Note that different distributions may apply to different climatic regions of the country. A brief discussion of several well-known distributions follows.

Three approaches are commonly used to distribute rainfall within a design storm. The first approach uses an average temporal distribution derived from hyetographs of actual storms. The second approach uses a simple temporal pattern (e.g., triangular) fitted to local storm data by the method of moments. The third approach uses a temporal pattern

derived from the local depth-duration-frequency relationship. Two excellent sources of information on the three approaches are ASCE (1983) and Harremöes (1983).

Average temporal patterns can be developed from local point-rainfall data if such data are available in short time intervals (15 min. or less). Such an analysis should consider only major storms, and storms should be grouped according to duration to account for different storm types. If an average temporal pattern is applied outside the region for which it was developed, caution and judgment must be used in interpreting the results.

Huff (1967) developed four average temporal patterns for heavy storms in east-central Illinois. Historical storms were divided into four groups according to the relative timing of the peak intensity, and average temporal patterns were developed for each group. The short-duration high-intensity storms that typically govern the designs of storm-drainage facilities were particularly common in the first-quartile group. Consequently, Terstriep and Stall (1974) recommend the first-quartile median distribution for urban design storms in Illinois. Huff's procedure has also been applied in Canada to derive average temporal patterns for one-hour and twelve-hour durations (Hogg 1980). These durations were selected to provide samples of both local convective storms and large-scale cyclonic storms. Hershfield (1962) presents average temporal patterns for longer-duration storms (6–24 hours).

An alternative approach is to use a simple idealized rainfall distribution fitted to local storm data by the method of moments. In this procedure, the fitting parameters of the idealized rainfall distribution are expressed in terms of the dimensionless moments of the distribution. The dimensionless moments of the idealized distribution are then set equal to the mean values of these dimensionless moments for major historical storms. The number of moments required equals the number of fitting parameters.

A simple triangular rainfall distribution, as shown in Figure 5.3, is often used in computing runoff hydrographs. A single fitting parameter, the time to the peak intensity, determines the shape of the distribution. This parameter, t_p, is related to the first moment of the hyetograph (the time to the centroid), $\bar{t}$, by the equation:

$$\frac{t_p}{t_d} = 3\frac{\bar{t}}{t_d} - 1 \qquad (5\text{-}3)$$

where t_d is the storm duration. Yen and Chow (1980) first used the method of moments to develop non-dimensional triangular hyetographs for various locations, durations, depths, and seasons. Times to peak on these triangular hyetographs range from 32–51% of the storm duration. However, most of the storms included in this study produced relatively small amounts of rainfall. McEnroe (1986) found much shorter times to peak by considering only major storms (in this case, storms that produced one-hour rainfalls with return periods of two years or

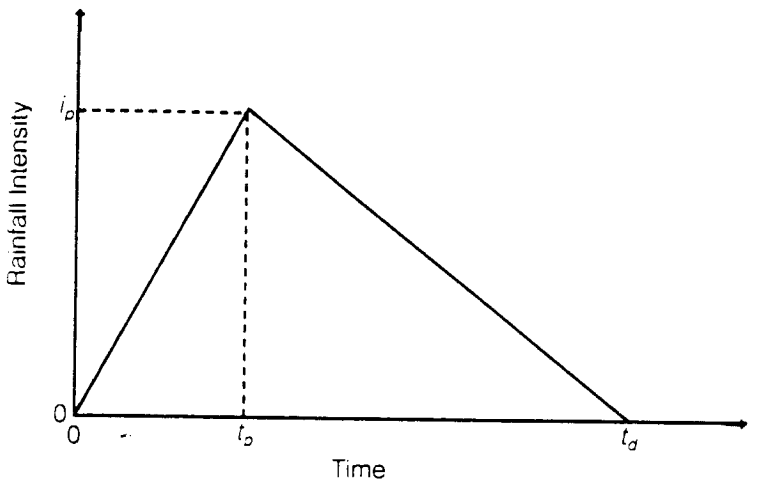

Figure 5.3—Triangular design storm.

greater). The following example illustrates the development of a triangular design storm.

Example 5–1: A 2-hour, 10-year triangular design storm is to be developed for Dodge City, Kansas. The 2-hour 10-year rainfall depth for this location is 2.80 inches. The mean value of t/t_d for intense storms in this region is 0.38 (McEnroe 1986).

Solution: Using Equation (5-3), one finds that the time to the peak rainfall intensity, t_p, is only 14 percent of the storm duration, or 17 minutes. The average rainfall intensity during the storm is 1.40 inches, the depth divided by the duration. The peak intensity, i_p is 2.80 inches per hour, twice the average intensity. These dimensions completely define the triangular design storm.

Two-parameter rainfall distributions permit one to adjust the peakedness of the storm as well as the timing of the peak intensity. A flexible two-parameter beta-function distribution has been proposed by Voorhees and Wenzel (1984). A two-parameter distribution used in Canada consists of a linear rise followed by an exponential decay, or vice versa (Marsalek and Watt 1983).

The third general approach for distributing rainfall within a design storm makes use of the local intensity-duration relationship for the design return period. This approach is based on the assumption that the maximum rainfall for any duration less than or equal to the total storm duration should have the same return period. For example, a 10-year three-hour design storm of this type would contain the 10-year rainfall depths for all durations from the shortest time interval considered (perhaps 5 minutes) up to three hours. These rainfalls are generally skewed. This distribution can be readily derived from the local IDF curves and the analysis of skewness of actual storms. A study of the conditional probabilities of intense rainfalls of different durations (Fred-

erick et al. 1977) indicates that this assumption is very conservative, particularly for longer storms.

Kiefer and Chu (1957) used this approach in the development of the so-called Chicago storm. Although the Chicago storm distribution is widely used in practice, a word of caution is in order. Recent extensive analyses in Canada (Hogg 1980) indicate that the Chicago-type distribution is totally inappropriate for some Canadian climates, and, for the bulk of the country, is not among the most probable distributions. A recently proposed modification of the Chicago distribution attempts to reduce the excessive sharpness of the storm hyetograph by averaging the storm segment that contains the intensity peak. The modified storm profile that results no longer contains all maximum rainfalls for some short durations.

The United States Soil Conservation Service (SCS) used this assumption in developing 24-hour design-rainfall distributions for four geographical regions in the United States (SCS 1986). The ratio of peak intensity to average intensity is higher for these SCS 24-hour design distributions than for most historical storms of this approximate duration.

One simple method for developing a design storm from intensity-duration-frequency data is termed the alternating-block method. This method is illustrated in the following example.

Example 5-2: Use the alternating-block method to develop a design storm with a 10-year return period and a 60-minute duration for St. Louis, Missouri. Use a 5-minute time interval. The 10-year intensity-duration relationship for St. Louis is described by the equation $i = 104.7 / (t_d^{0.89} + 9.4)$ where i is the average rainfall intensity in inches per hour and t_d is the duration in minutes (Wenzel 1982).

Solution: First, average rainfall intensities are computed for durations from 5–60 minutes in 5-minute increments. These values are shown in column 2 of Table 5.2. Cumulative depth, shown in column 3, is the product of duration and average intensity. The incremental depths in column 4 are the differences between successive values of cumulative depth. These values are the 5-minute rainfall amounts, arranged in descending order. These rainfall amounts are expressed as average intensities in column 2. The period of highest intensity is assumed to occur just before the midpoint of the 60-minute storm duration. The other 5-minute blocks of rainfall are arranged in descending order alternately to the right and left of the largest block, as shown in Figure 5.4. With this arrangement, the maximum rainfall for any duration from 5–60 minutes has a 10-year return period.

For practical applications, it is recommended to use one of the existing standard distributions approved by the client. Where the designer must develop the temporal distribution, in the absence of comprehensive evaluations and comparisons, it is recommended to use the simpler ones, such as the triangular or combined linear/exponential distribu-

TABLE 5.2: Development of Alternating-Block Design Storm

Duration min. (1)	Average Intensity in./hr (2)	Cumulative Depth in. (3)	Incremental Depth in. (4)	Incremental Intensity in./hr (5)
5	7.68	0.640	0.640	7.68
10	6.09	1.015	0.375	4.49
15	5.09	1.272	0.257	3.09
20	4.39	1.463	0.191	2.31
25	3.88	1.617	0.154	1.82
30	3.48	1.740	0.123	1.49
35	3.16	1.843	0.103	1.25
40	2.90	1.933	0.090	1.07
45	2.68	2.010	0.077	0.93
50	2.50	2.083	0.073	0.82
55	2.34	2.145	0.062	0.73
60	2.20	2.200	0.055	0.66

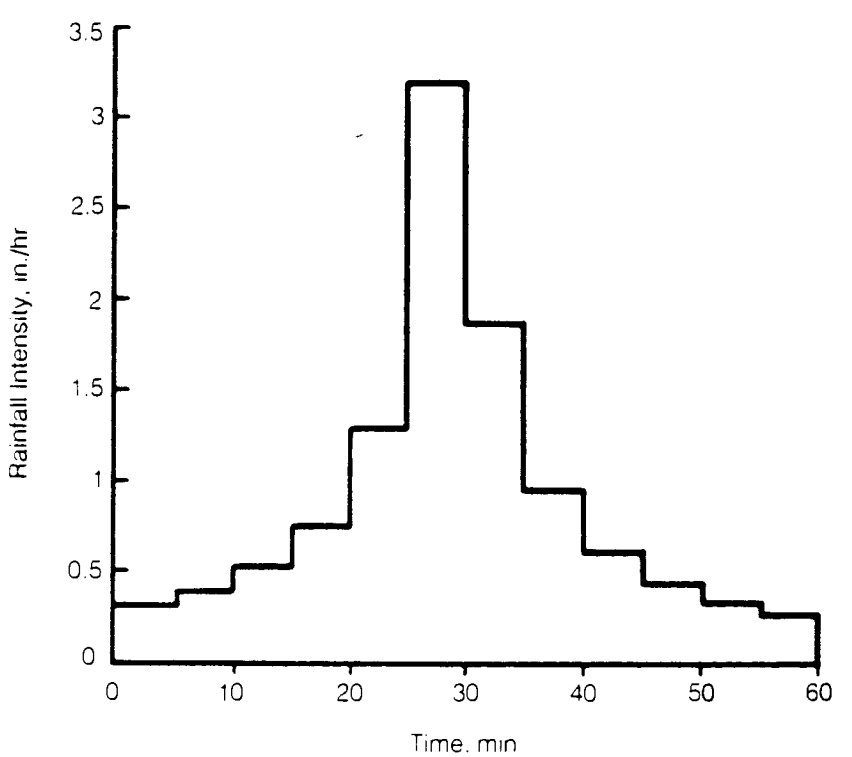

Figure 5.4—Alternating-block design storm (in. × 25.4 = mm).

tions. The fitting of these conceptual models to actual rainfall data is done by the method of moments. The goodness of fit has to be further tested. The selected distribution is then applied to the total rainfall to produce the storm hyetograph.

4. *Spatial Distribution*

Storm spatial characteristics are important for larger catchments. In general, the larger the catchment and the shorter the rainfall duration, the less uniformly the rainfall is distributed over the catchment. For any specified return period and duration, the average rainfall depth over an area is less than the point rainfall depth. The ratio of the areal average rainfall with a specified duration and return period to the point rainfall with the same duration and return period is termed the areal reduction factor. The areal reduction factor can be estimated using Figure 5.5, developed by the National Weather Service (Miller et al. 1973).

Once the areal average rainfall has been determined, this rainfall is normally assumed to be distributed uniformly over the catchment. Various models have been developed for simulating the effects of storm dynamics on the temporal and spatial distribution of rainfall over a catchment. Storm direction and movement can have marked effects, particularly in areas with predominating weather patterns, and are particularly relevant to the case of operation and/or control of a large system of combined sewers (James and Drake 1980).

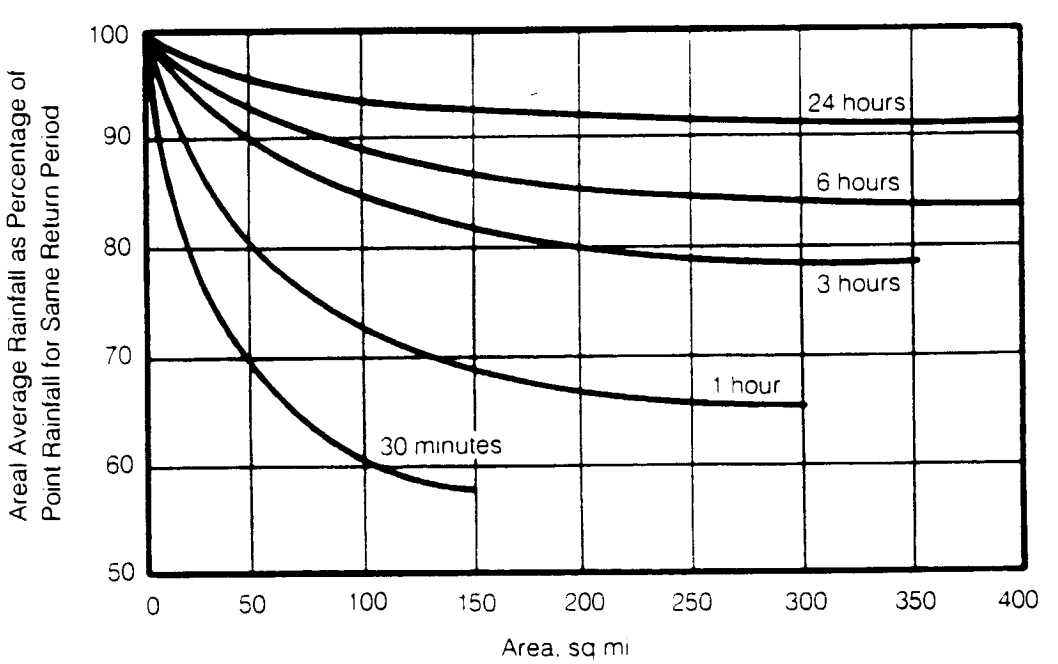

Figure 5.5—Ratio of areal average rainfall to point rainfall for a constant return period (sq mi × 2.590 = km²) (Miller et al, *1973).*

Although temporal distributions are often expressed as continuous functions, for actual use in runoff models they must be discretized into time intervals generally coinciding with the computational time step.

In light of the uncertainties inherent in the selection of other design inputs, and the general unavailability of the detailed rainfall data needed for calibration, modelling of storm dynamics would appear to be an unnecessary refinement at this time.

5. *Other Methods for Design Hydrology*

Another technique for arriving at design rainfall is to analyze statistically the long-term rainfall record for a station in or near the catchment, to determine the return frequencies of actual rainfall events. Appropriate events can then be used as inputs to runoff calculations. Alternatively, **actual** storms that produce rainfall equivalent to the "design" storm can be used. The graphics capabilities of modern computers provide a quick and easy means of making those comparisons (also see the following section on Infiltration). Marsalek (1978), Wenzel (1982), and Bedient and Huber (1988) should be consulted for further information on actual storms.

Frequency analysis of synthetic runoff records generated by continuous simulation is another alternative to the use of design storms (McPherson 1978). It is particularly useful when flow volumes are of concern (as when designing detention/retention storage). Despite the fact that synthetic runoff records may not be entirely "accurate" unless the continuous simulation model has been calibrated using local data, the method allows the selection of **actual** rainfall events that have produced **runoff** of some specified return frequency.

E. Rainfall Abstractions

The physical processes of interception of rainfall by vegetation, infiltration of water into the soil surface, and storage of water in surface depressions are commonly termed rainfall abstractions. Although these three processes are physically complex, some simplified modelling procedures have been found acceptable in urban areas. Evaporation is generally insignificant during the short-duration storms of concern in storm drainage design. The portion of the rainfall that is not abstracted by interception, infiltration, or depression storage is termed the excess rainfall.

1. *Interception*

The amount of rainfall intercepted by vegetation depends on vegetation type, growth stage, wind speed, and rainfall intensity and duration. Interception is averaged over the surface area and expressed as a depth. The data of Horton (1919) and others show that the interception storage capacity of vegetation can range from less than 0.01–0.5 inches. A typical value for grass turf is 0.05 inches.

2. *Infiltration*

The infiltration process plays the dominant role in determining storm runoff from pervious surfaces. The primary factors affecting rainfall infiltration are the initial soil wetness, the temporal distribution of rainfall intensity, the soil type, the vegetative cover, and special considerations such as whether the soil surface is crusted or frozen. At any instant, the maximum rate at which the soil surface can absorb water is termed the potential infiltration rate. At the beginning of a storm, all rainfall reaching the soil surface is absorbed into the soil. However, the potential infiltration rate decreases as the total infiltrated depth increases. This is caused by a reduction in the hydraulic gradient at the surface, and in some cases also by surface sealing, crusting, or other factors. When the rate at which rainfall reaches the soil surface exceeds the potential infiltration rate, water ponds on the soil surface and runoff begins.

Several approximate methods are widely used by engineers to estimate infiltration losses for storm rainfall. These include the equations of Green and Ampt, Horton, and the U.S. Soil Conservation Service (SCS), which are discussed below. Other approximate methods sometimes used in watershed modelling include the Holtan equation and the exponential loss-rate function of the U. S. Army Corps of Engineers' Hydrologic Engineering Center (HEC). More sophisticated methods of infiltration analysis are based on numerical solution of the complete equations governing unsaturated vertical flow in soils. These methods are almost never used in engineering practice on the watershed scale because of their computational complexity and because of the difficulties associated with obtaining the necessary inputs.

3. *Green-Ampt Equation*

The Green-Ampt equation is an approximate infiltration equation based on Darcy's law (Green and Ampt 1911). Infiltrated water is assumed to move downward through the soil with an abrupt wetting front separating the wetted and unwetted zones. At any instant the potential infiltration rate, f_p, is given by the equation:

$$f_p = K\left[1 + \frac{\Psi_f(\phi - \theta_i)}{F}\right] \tag{5-4}$$

where K is the hydraulic conductivity of the transmission zone, Ψ_f is the capillary suction head at the wetting front, ϕ is the porosity of the soil (the volumetric water content at saturation), θ_i is the initial volumetric water content of the soil, and F is the cumulative infiltration (the total amount of infiltration that has occurred since rainfall began). Ponding begins when the potential infiltration rate equals the rainfall rate. The cumulative infiltration at any time during ponding is given by the equation:

$$F - F_p - \Psi_f(xH - \theta_i)\, ln\left[\frac{F + \Psi_f(\phi - \theta_i)}{F_p + \Psi_f(\phi - \theta_i)}\right] = K(t - t_p) \qquad (5\text{-}5)$$

where t_p is the time at the onset of ponding, F_p is the cumulative infiltration at time t_p, and F is the cumulative infiltration at time t.

The Green-Ampt infiltration equation has several advantages over other commonly-used infiltration equations. Some of these advantages are its physical basis, its explicit consideration of initial soil water content, and its direct applicability to conditions of unsteady rainfall. The values of the Green-Ampt parameters can be estimated from physical soil characteristics and knowledge of the initial moisture content (for θ_i). Mein and Larsen (1973), present the easiest and most often cited method of using the Green-Ampt equation.

The values of the Green-Ampt parameters for a particular soil can be estimated in several different ways. Procedures have been developed for estimating the values of the parameters K and Ψf based on the soil's porosity, saturated hydraulic conductivity, and water retention characteristics (Bouwer, 1966; Brakensiek and Onstad, 1977). Alternatively, the Green-Ampt equation can be fitted to in-situ infiltrometer data (Brakensiek and Onstad, 1977). Table 5.3 presents some average values of the Green-Ampt parameters sorted according to soil texture class. These values are based on statistics for porosity, saturated hydraulic conductivity, and water-retention parameters for some 5000 soil horizons compiled by the U.S. Agricultural Research Service (Rawls et al. 1983). Actual values of these parameters vary widely within a soil texture class. The values in Table 5.3 should not be used for bare soils

TABLE 5.3: Average Values of Green-Ampt Parameters by USDA Soil-Texture Class (Rawls et al. 1983)

USDA Soil-Texture Class (1)	Hydraulic Conductivity K_1 in/hr (2)	Wetting-Front Suction Head ψ_{f1} in (3)	Porosity in³/in³ (4)	Water Retained @ Field Capacity in³/in³ (5)	Water Retained @ Wilting Point in³/in³ (6)
Sand	4.74	1.93	0.437	0.062	0.024
Loamy Sand	1.18	2.40	0.437	0.105	0.047
Sandy Loam	0.43	4.33	0.453	0.190	0.085
Loam	0.13	3.50	0.463	0.232	0.116
Silt Loam	0.26	6.69	0.501	0.284	0.135
Sandy Clay Loam	0.06	8.66	0.398	0.244	0.136
Clay Loam	0.04	8.27	0.464	0.310	0.187
Silty Clay Loam	0.04	10.63	0.471	0.342	0.210
Sandy Clay	0.02	9.45	0.430	0.321	0.221
Silty Clay	0.02	11.42	0.479	0.371	0.251
Clay	0.01	12.60	0.475	0.378	0.265

with crusted surfaces. This table also provides some typical values of water content at field capacity (pore-water pressure of $-1/3$ bar, a typical value after prolonged gravity drainage) and wilting point (pore-water pressure of -15 bars, roughly the limiting value for transpiration) to guide the selection of an initial water content. Table 5.3 provides a reasonable basis for engineering estimates of infiltration on the watershed scale.

The following example shows how the Green-Ampt equation is used to compute infiltration losses for constant-intensity rainfall. The case of variable-intensity rainfall is covered in detail by Chow et al. (1988).

Example 5-3: A 30-minute rainfall with a constant intensity of 5 cm/hr occurs on a typical silt-loam soil that is initially at field capacity. The objective is to determine the time to the onset of ponding and the total amount of runoff produced.

Solution: The values of the Green-Ampt parameters and the water content at field capacity for this soil are assumed to be the mean values for soils in this texture class, from Table 5.2: $K = 0.65$ cm/hr, $\Psi_f = 17$ cm, $\phi = 0.501$, and $\theta_i = 0.284$. The cumulative infiltration at the onset of ponding, F_p, is found using equation (5-4) by setting f_p, the potential infiltration rate, equal to the rainfall rate. This yields a value of 0.55 cm for F_p, indicating that ponding begins when 0.55 cm of infiltration has occurred. Until ponding begins, the actual infiltration rate equals the rainfall rate, so the time to the onset of ponding, t_p, is 0.11 hours, or 7 minutes. The cumulative infiltration at any time during ponding can be found using equation (5-5). Setting t equal to 30 minutes, the rainfall duration, and solving for F by trial, one obtains a total infiltration of 1.67 cm. The total runoff is 0.83 cm, the difference between the 2.50 cm of rainfall and the 1.67 cm of infiltration.

Note that rainfall interception is neglected in this example for the sake of simplicity. In practice, one should make reasonable estimates of these losses.

4. *Horton Equation*

The Horton infiltration equation (Horton, 1939 and 1940) is an empirical three-parameter equation that describes the decrease in infiltration rate over time when water is ponded on a soil surface. The Horton equation is:

$$f = f_c + (f_o - f_c)e^{-kt} \tag{5-6}$$

where f is the infiltration rate, f_o is the maximum or initial infiltration rate, f_c is the minimum or ultimate infiltration rate that is approached

t is the elapsed time since infiltration began, and k is a constant that reflects how rapidly the infiltration rate decreases with time.

Equation (5-6) is not directly applicable to storm rainfall applications. The potential infiltration rate will not actually decrease as rapidly as indicated by this equation unless the rainfall intensity always exceeds the infiltration rate. This problem is sometimes sidestepped by considering the potential infiltration rate to be a function of cumulative infiltration rather than elapsed time (see Viessman et al. 1989, for an excellent discussion). This form of the Horton equation is used in the U.S. Environmental Protection Agency's Storm Water Management Model (SWMM). The relationship between potential infiltration rate and cumulative infiltration is assumed to be identical to the relationship between actual infiltration rate and cumulative infiltration under ponded conditions. This assumption leads to a rather awkward procedure for applying the Horton infiltration equation to steady and unsteady rainfalls. This procedure is presented in detail by Chow et al. (1988).

Since the Horton equation is strictly empirical, the values of its three parameters should be determined by fitting the equation to field or laboratory infiltrometer data. These fitted values are sensitive to the antecedent wetness of the soil. No reliable procedure exists for estimating the values of the Horton parameters from the physical characteristics of a soil, or for adjusting these values to account for initial soil wetness or vegetative cover.

5. *Soil Conservation Service Equation*

The well known rainfall-runoff relationship of the U.S. Soil Conservation Service (SCS 1972) is often misused as an infiltration equation. The SCS procedure, often referred to as the curve-number procedure, was originally developed to provide an estimate of total runoff volume for a storm event based on total storm rainfall, antecedent watershed conditions, an SCS soil classification, and surface cover conditions. Derived from storms of 24-hour or less duration on small to very small rural watersheds, this empirical equation does not consider the effect of rainfall intensity on runoff volume. Differentiating the SCS rainfall-runoff equation with respect to time yields a rate equation. This rate equation indicates that the infiltration rate is directly proportional to the rainfall intensity, a relationship that is unacceptable from a physical standpoint. For this reason, use of the SCS rainfall-runoff relationship as an infiltration equation is discouraged.

6. *Depression Storage*

Depression storage refers to water that accumulates in surface depressions during a storm. This water eventually evaporates or infiltrates following the storm. Like infiltration, depression storage is expressed as an average depth over the catchment area. Typical depths of depres-

sion storage are 0.05 to 0.10 inches for impervious surfaces such as pavements and roofs (Tholin and Kiefer 1960), 0.10–0.20 inches for lawns (Hicks 1944), 0.2 inches for pasture, and 0.3 inches for forest litter. These typical values are for surfaces with moderate slopes. Typical values would be larger for flat slopes and smaller for steep slopes. The manual for the SWMM is a good source of European depression storage data (Huber and Dickinson 1988).

F. Runoff Hydrographs

Many different procedures are used to compute runoff hydrographs for storm events on small urban catchments. Most of these procedures make use of synthetic unit hydrographs, time-area curves, or kinematic-wave analysis. This section discusses these three general methods, plus the nonlinear-reservoir procedure used in the SWMM. A more detailed treatment of each of these methods can be found in the references provided, and in standard textbooks on engineering hydrology.

1. Synthetic Unit Hydrograph

A unit hydrograph is a hydrograph resulting from an excess rainfall of unit depth with a specified duration and a fixed spatial and temporal pattern. Hydrographs for complex historical and design storms can be computed using the unit hydrograph. Unit hydrograph theory assumes that a linear transfer function relates the excess rainfall on the catchment to the stream discharge at the catchment outlet. This assumption is often a reasonable and useful approximation. Unit hydrographs should be developed from actual streamflow and rainfall data if such data are available. Unit hydrograph theory and procedures, including derivation of unit hydrographs from actual streamflow and rainfall data, are discussed in detail in engineering hydrology texts.

In the analysis and design of storm drainage facilities, developing unit hydrographs from field data is often impossible or impractical. Instead, a synthetic unit hydrograph procedure is used. Synthetic unit hydrograph procedures relate unit hydrograph characteristics to watershed and channel characteristics. Snyder (1938) developed the first synthetic unit hydrograph procedure using data for Appalachian Highland watersheds with areas of 10 mi^2 to 10,000 mi^2. Snyder's basic procedure has been applied by others to different regions and watershed types.

One adaptation of Snyder's procedure is a 10-minute unit hydrograph for urban areas developed by Espey and Altman (1978). The Espey 10-minute unit hydrograph was developed from data for 41 watersheds throughout the United States with areas ranging from 10 acres to 15 mi^2 and impervious fractions ranging from 2–100 percent. The dimensions of the Espey 10-minute unit hydrograph depend upon five watershed characteristics: the drainage area, the percentage of the total surface area that is impervious, the length of the main drainage channel

from the study point to the watershed divide, the average slope of the main channel, and the average Manning n value of the main channel. The following parameters were selected to describe the shape of the hydrograph (see also Figure 5.6):

T_R = Time of rise, minutes.
Q = Peak discharge, cfs.
T_B = Time base, minutes.
W_{50} = Time, in minutes, between the two points on the unit hydrograph at which the discharge is half the peak discharge.
W_{75} = Time, in minutes, between the two points on the hydrograph at which the peak discharge is three-fourths of the peak discharge.

The Colorado Urban Hydrograph Procedure (CUHP) (Urban Drainage and Flood Control District 1984) is another adaptation of Snyder's procedure based on data for Colorado urban watersheds ranging in size from 100–2000 acres. The dimensions of the CUHP hydrograph depend upon the unit excess-rainfall duration and five watershed characteristics: the drainage area, the percentage of the total surface area that is im-

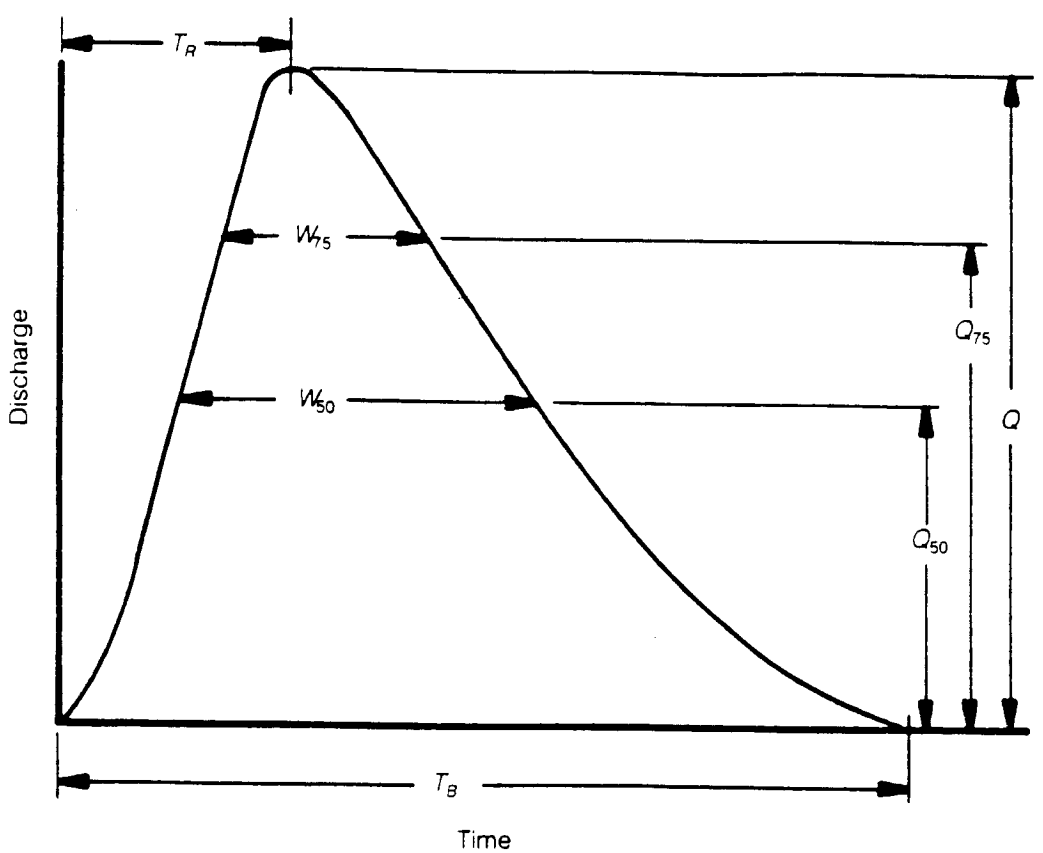

Figure 5.6—Definition of unit hydrograph parameters (Espey and Altman, 1978).

pervious, the length of the main drainage channel from the study point to the watershed divide, the distance along the main drainage channel from the study point to a point adjacent to the centroid of the watershed, and a weighted-average slope of the main drainage channel.

The dimensionless synthetic unit hydrographs of the U.S. Soil Conservation Service are also often applied to urban watersheds. These two dimensionless synthetic hydrographs, one triangular and the other curvilinear, are shown in Figure 5.7. To convert one of the dimensionless hydrographs to dimensional form, one needs values for the peak discharge, Q_p, and the time to peak, t_p. The time to peak is given by the formula:

$$t_p = 0.5t_r + 0.6t_c \tag{5-7}$$

where t_r is the unit excess-rainfall duration and t_c is the time of concentration for the watershed. Estimation of the time of concentration is discussed in Section G. The SCS recommends that t_r not exceed two-tenths of t_c. The time base, t_b, equals $2.67t_p$ for the triangular unit hydrograph and $5t_p$ for the curvilinear unit hydrograph. Both t_p and t_b should be rounded to the nearest whole multiple of t_r. From its simple ge-

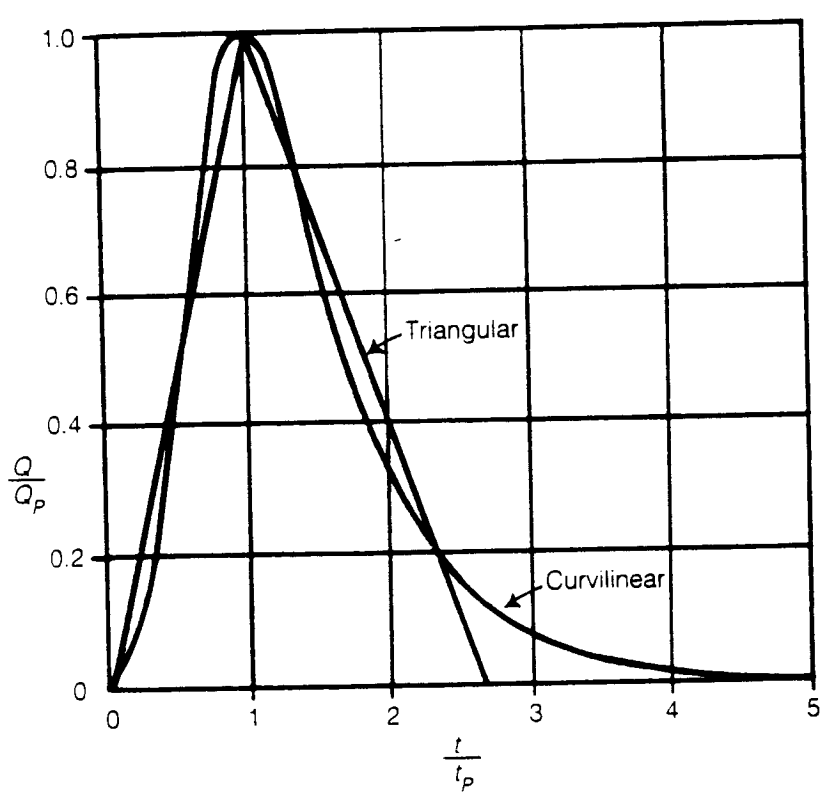

Figure 5.7—U.S. Soil Conservation Service (SCS) dimensionless synthetic unit hydrographs (U.S. Soil Conservation Service, 1986).

ometry, the peak discharge on the triangular unit hydrograph is given by the formula:

$$Q_p = \frac{2C\,D\,A}{t_b} \tag{5-8}$$

where A is the drainage area, D is the unit depth of excess rainfall, and C is a units-conversion constant. When D equals one inch and Q_p is in cfs, A is in acres, and t_p is in minutes, C has a value of 60.5. When D equals one centimeter and Q_p is in m^3/s, A is in hectares and t_b is in minutes, C has a value of 1.67. The values of t_p and Q_p computed using Equations (5-7) and (5-8) can also be applied to the curvilinear dimensionless unit hydrograph.

Example 5-4: Use the SCS procedure to develop a triangular synthetic unit hydrograph for a 250-acre watershed with a time of concentration of 30 minutes.

Solution: Because the unit excess-rainfall duration should not exceed two-tenths of the time of concentration, a 5-minute unit hydrograph is selected. A peak discharge and time to peak are needed to define the unit hydrograph. Equation (5-7) gives a time to peak of 20.5 minutes. The corresponding time base is 54.7 minutes. After rounding to the nearest whole multiple of t_r, t_p equals 20 minutes and t_b equals 55 minutes. From equation (5-8), the peak discharge is 550 cfs. Using these values of t_p and Q_p, the dimensionless unit hydrograph of Figure 5.7 can be converted to dimensional form.

2. *Time-Area Curves*

A particularly simple type of synthetic unit hydrograph is the time-area histogram. The time-area routing method is used in the Illinois Urban Drainage Area Simulator (ILLUDAS) computer program (Terstriep and Stall 1974). This method neglects all storage effects in the catchment. This is a reasonable approximation in many, but not all, urban catchments. The time-area routing method is a simplified version of Clark's method for watershed routing (Clark 1945). In Clark's method, the hydrograph developed using the time-area histogram is routed through a hypothetical linear reservoir to account for the effects of storage in the catchment.

Development of a time-area histogram requires a map of the catchment showing isochrones, lines of constant travel time to the catchment outlet. These isochrones are constructed for whole multiples of the time increment used in the infiltration and routing calculations. Travel-time should be the wave travel time (see discussion in Section II G, rational method), though estimates have often been based on velocities for uniform flow computed using Manning's equation. The incremental

areas between adjacent isochrones are determined and plotted against travel time in the form of a histogram. This histogram represents the synthetic unit hydrograph for the catchment. For catchments that contain both pervious and impervious surfaces, it is advantageous to develop two separate time-area relationships: one for the directly connected impervious area and another for the remaining area. An advantage of the time-area method is its ability to account for unusual catchment shapes and nonuniform spatial distributions of excess rainfall.

3. *Kinematic Wave*

The kinematic-wave method is a hydraulic method for routing runoff across planar surfaces and through small channels and pipes. The kinematic-wave formulation couples the continuity equation with a simplified form of the momentum equation that includes only the bottom-slope and friction-slope terms. This is usually a valid approximation provided that backwater effects are not present. In modelling overland flow over a pervious surface, the problem formulation also includes an infiltration equation.

The kinematic-wave equations are solved numerically by a finite-difference method or by the method of characteristics. Analytical solutions are available for some special cases, such as for overland flow with constant rates of rainfall and infiltration (Eagleson 1970). The widely used HEC-1 flood hydrograph program of the Army Corps of Engineers contains options for kinematic-wave routing of overland and channel flows. This application of kinematic-wave procedures is discussed in the HEC-1 User's Manual (Hydrologic Engineering Center 1985) and a related background document (Hydrologic Engineering Center 1979). The kinematic wave method for urban stormwater simulation is also used in the U.S. Geological Survey's Distributed Routing Rainfall Runoff Model (DR3M) (Alley and Smith 1982), and in the SWMM Transport Block.

Kinematic-wave theory provides a useful formula for the time of concentration for one-dimensional overland flow on a planar surface. This time of concentration is defined as the time required for equilibrium discharge (outflow = inflow) to become established at the point of interest. For a constant excess rainfall rate, this time of concentration, t_c, is given by the formula:

$$t_c = C \frac{n^{0.6} L^{0.6}}{i_e^{0.4} S^{0.3}} \tag{5-9}$$

in which L is the distance from the upper end of the plane to the point of interest, n is the Manning resistance coefficient, i_e is the excess-rainfall rate, S is the dimensionless slope of the surface, and C is a constant that depends on the units of the other variables. For t_c in minutes, i_e in in./hr, and L in feet, C equals 0.938. For t_c in minutes, i_e in mm/hr, and L in meters, C equals 6.99. This equation is for tur-

bulent flow and for use of Manning's equation (for a more general equation, see Eagleson (1970)). Estimates of Manning's n for overland flow are shown in Table 5.4. It should be noted that the kinematic wave velocity for turbulent overland flow using Manning's equation is 5/3 of the water velocity. Thus a t_c found by conventional methods (based on water parcel travel time) could be adjusted by multiplying by 5/3.

4. *Nonlinear Reservoir*

In the nonlinear reservoir method, the catchment is conceptualized as a very shallow reservoir. The discharge from this hypothetical reservoir is assumed to be a nonlinear function of the depth of water in the reservoir. The SWMM uses a nonlinear reservoir approach to compute surface runoff hydrographs. The SWMM procedure is described here (Huber and Dickinson 1988).

Figure 5.8 shows the catchment conceptualized as a reservoir with rainfall as inflow, and infiltration and surface discharge as outflows. The depth y represents the average depth of surface runoff, and the depth y_d represents the average depression storage in the catchment. The continuity relationship for this system is:

$$A \frac{dy}{dt} = A(i - f) - Q \tag{5-10}$$

where A is the catchment area, i is the rainfall intensity, f is the infiltration rate, and Q is the discharge at the catchment outlet. The model assumes uniform overland flow at the catchment outlet at a depth equal

TABLE 5.4: Estimates of Manning's n for Overland Flow

Surface Type (1)	Manning's n (2)	Range (3)
Concrete/Asphalt**	0.011	0.01–0.013
Bare Sand**	0.01	0.01–0.016
Bare Clay—Loam (eroded)**	0.02	0.012–0.033
Gravelled Surface**	0.02	0.012–0.03
Packed Clay*	0.03	
Short Grass Prairie**	0.15	0.10–0.20
Light Turf*	0.20	
Lawns*	0.25	0.20–0.30
Dense Turf*	0.35	
Pasture*	0.35	0.30–0.40
Dense Shrubbery and Forest Litter*	0.40	
Bluegrass Sod**	0.45	0.39–0.63

*From Crawford and Linsley (1966)—obtained by calibration of Stanford Watershed Model.
**From Engman (1986) by kinematic wave and storage analysis of measured rainfall-runoff data.

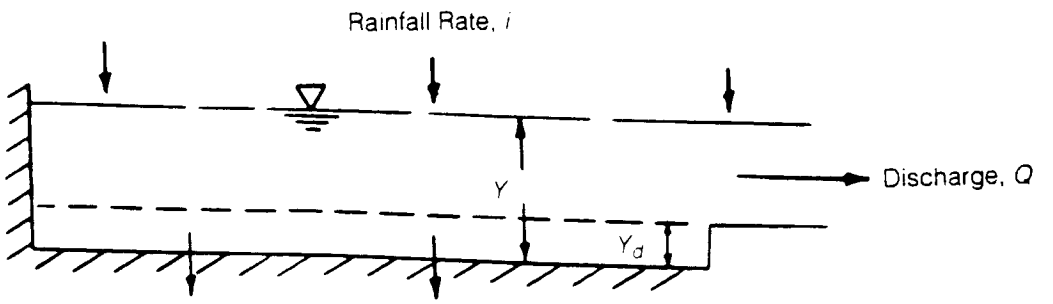

Figure 5.8—Definition sketch for nonlinear-reservoir model.

to the difference between y and y_d. Based on the Manning friction relationship, the catchment discharge, Q, is given by:

$$Q = \frac{CW}{n}(y - y_d)^{5/3}S^{1/2} \tag{5-11}$$

where C is a constant, W is a representative width for the catchment, n is an average value of the Manning roughness coefficient for the catchment, and S is an average surface slope. The constant C has a value of 1.49 when y and y_d are in feet and Q is in ft^3/s, and a value of 1 when y and y_d are in meters and Q is in m^3/s. Substituting equation (5-11) into equation (5-10) yields a nonlinear differential equation for y. A simple finite difference form of the equation is used to solve for the depth y at the end of each time step. This equation is:

$$\frac{y_2 - y_1}{\Delta t} = \bar{i} - \bar{f} - \frac{CWS^{1/2}}{An}\left(\frac{y_1 + y_2}{2} - y_d\right)^{5/3} \tag{5-12}$$

where Δt is the time-step increment, y_1 is the depth at the beginning of the time step, y_2 is the depth at the end of the time step, and $\bar{i}$ and $\bar{f}$ are the average rainfall and infiltration rates over the time step.

For each time step, three separate calculations are performed. First, an infiltration equation is used to compute the average potential infiltration rate over the time step (in the SWMM program, the user selects either the Green-Ampt or the Horton infiltration equation), then equation (5-12) is solved iteratively for y_2, and, finally, equation (5-11) yields the corresponding discharge.

Unlike the synthetic unit hydrograph and time-area methods, which use excess rainfall as input, the nonlinear-reservoir method couples the processes of infiltration and surface runoff. The nonlinear-reservoir model assumes that infiltration occurs at the potential rate over the entire surface area whenever the ponded depth is non-zero. The excess-rainfall models, on the other hand, entirely neglect infiltration of ponded water. This difference becomes important following cessation of rainfall, or

whenever the rainfall intensity drops below the potential infiltration rate. In reality, infiltration does continue for some time after rainfall ceases, but the area over which infiltration continues to occur decreases rapidly with time. Thus the runoff that occurs after rainfall ceases would tend to be underestimated by the nonlinear-reservoir methods and overestimated by the excess-rainfall methods, though the difference will depend on the degree of discretization used, since a more detailed schematization can partially account for the phenomenon of decreasing area of infiltration.

G. Rational Method For Peak Discharge

In the rational method, known as the Lloyd-Davies method in the United Kingdom, peak discharge is related to rainfall intensity by the formula:

$$Q = CiA \tag{5-13}$$

in which Q is the peak discharge, in cfs; C is a non-dimensional runoff coefficient; i is the average rainfall intensity, in in./hr, over a duration equal to the time of concentration for the contributing area; and A is the contributing area, in acres (1 acre-inch/hr = 1.008 cfs).

The rational method is based on the following assumptions:

(a) The peak discharge at any point is directly proportional to the average rainfall intensity during the time of concentration to that point.
(b) The return period of the peak discharge is the same as the return period of the average rainfall intensity.
(c) The time of concentration is the travel time from the most remote point in the contributing area to the point under consideration. This assumption applies to the point most remote in time, not necessarily in distance.
(d) The contributing area can be the entire drainage area upstream of the design point or some subset of this area, such as only the directly connected impervious portion of the drainage area.

1. Limitations of the Rational Method

Experience has shown that **when applied properly**, the rational method can provide satisfactory estimates for **peak** discharges on small catchments where storage effects are insignificant. The rational method is not recommended for drainage areas much larger than 100–200 acres, for any catchment where ponding of stormwater in the catchment might affect peak discharge, or where the design and operation of large (and hence more costly) drainage facilities is to be undertaken, particularly if they involve storage.

It may be possible to use the rational method in some cases where ponding affects peak discharge, by adjusting the runoff coefficient to compensate for the ponding. While procedures have been developed

for the use of the rational method in detention basin design (see Chapter 11), care should be exercised since the rational method does not produce a discharge hydrograph (Adams and Howard 1986). Procedures that use the rational method as a basis for constructing pseudo-hydrographs are not recommended.

2. *Runoff Coefficient*

The runoff coefficient, C, accounts for the integrated effects of rainfall interception, infiltration, depression storage, and temporary storage in transit on the peak rate of runoff. When estimating a value for the runoff coefficient, the engineer should consider the roles played by these hydrologic processes. The runoff coefficient depends on rainfall intensity and duration as well as the catchment characteristics. The greater the rainfall depth, the lesser the relative effect of rainfall abstractions on the peak discharge, and therefore the greater the runoff coefficient.

Decades of practical experience with the rational formula have led to some accepted ranges of values for the runoff coefficient. Table 5.5 shows normal ranges of values for several types of surfaces. Table 5.6 shows some typical ranges for the composite runoff coefficient for various urban land uses. In practice, the composite runoff coefficient for a heterogeneous catchment should be computed as the area-weighted average of the runoff coefficients for the different types of surfaces in the catchment.

The ranges of values in Tables 5.5 and 5.6 are typical for return periods of 2–10 years. Higher values are appropriate for longer return

TABLE 5.5: Normal Range of Runoff Coefficients*

Character of Surface (1)	Runoff Coefficients (2)
Pavement	
Asphalt and Concrete	0.70 to 0.95
Brick	0.70 to 0.85
Roofs	0.75 to 0.95
Lawns, Sandy Soil	
Flat (2 percent)	0.05 to 0.10
Average (2 to 7 percent)	0.10 to 0.15
Steep (>7 percent)	0.15 to 0.20
Lawns, Heavy Soil	
Flat (2 percent)	0.13 to 0.17
Average (2 to 7 percent)	0.18 to 0.22
Steep (>7 percent)	0.25 to 0.35

*The range of "C" values presented are typical for return periods of 2–10 years. Higher values are appropriate for larger design storms.

TABLE 5.6: Typical Composite Runoff Coefficients, by Land Use*

Description of Area (1)	Runoff Coefficients (2)
Business	
Downtown	0.70 to 0.95
Neighborhood	0.50 to 0.70
Residential	
Single Family	0.30 to 0.50
Multi-units, detached	0.40 to 0.60
Multi-units, attached	0.60 to 0.75
Residential (suburban)	0.25 to 0.40
Apartment	0.50 to 0.70
Industrial	
Light	0.50 to 0.80
Heavy	0.60 to 0.90
Parks, cemeteries	0.10 to 0.25
Playgrounds	0.20 to 0.35
Railroad yards	0.20 to 0.35
Unimproved	0.10 to 0.30

*The ranges of "C" values presented are typical for return periods of 2–10 years. Higher values are appropriate for larger design storms.

periods, because infiltration and other losses then have a proportionally smaller effect on runoff.

3. *Rainfall Intensity*

The rainfall intensity, i, in the rational formula represents the average rainfall intensity over a duration equal to the time of concentration for the catchment. This combination of average intensity and duration must have a return period equal to the desired return period of the peak discharge. This rainfall intensity therefore depends on the following: (1) the desired return period of the peak discharge, (2) the local rainfall depth-duration relationship for this return period, and (3) the time of concentration for the catchment. Sources of rainfall depth-duration-frequency data were discussed in Section D.

4. *Time of Concentration*

The time of concentration is the travel time of a **wave** from the most hydraulically remote point in the contributing area to the point under study (it should be noted that many references define time of concentration as the travel time of a parcel of water to move down the catchment). This can be considered the sum of an overland-flow time and times of travel in street gutters, roadside swales, storm sewers, drainage

channels, small streams, and other drainageways. The major factors affecting time of concentration for overland flow are maximum flow distance, surface slope, surface roughness, rainfall intensity, and infiltration rate. The wave speed in a gutter, pipe, or channel at a particular discharge can be calculated using equation (5-14):

$$c = Vt \pm \sqrt{\frac{gA}{B}} \tag{5-14}$$

where

V = water velocity
g = gravitational acceleration
A = cross-sectional area of flow
B = top width of flow

For subcritical flow, the wave of interest is in the downstream direction, for which the plus sign is used. Here, c is obviously $> V$, and, once again, the t_c based on wave speed is less than t_c based on water velocity. In the design of storm drainage systems, the time of concentration is often considered the sum of the time of travel to an inlet and the time of travel in the storm sewer or drainage channel. The inlet time usually includes both overland flow time and flow time in gutters or roadside swales.

For overland flow, wave speed is usually given by the kinematic wave equation, and the time of concentration for overland flow by kinematic wave theory is given by equation (5-9). Since the t_c equation (5-9) involves the use of rainfall excess, i_e, which is not a constant, the calculation of time of concentration requires iteration between the IDF curve and the kinematic wave equation for t_c until t_c = rainfall duration. Excellent discussions of this issue can be found in Bedient and Huber (1988) and Eagleson (1970). It should be noted that any of the popular microcomputer spreadsheet programs would be very useful in such calculations.

Many other equations, mostly empirical, have been proposed for estimating times of concentration for urban watersheds. For watersheds with varied land use and significant channel-flow time, no single equation can be expected to provide an accurate estimate of time of concentration. In such cases, the recommended procedure is to break the flow path into segments that are somewhat homogeneous and then use a velocity-based equation to estimate the flow time in each segment. It is common practice to assume an arbitrary minimum time of concentration that has been observed to range, depending on local practice, from 5–20 minutes.

In any method for calculating times of concentration, uncertainties about the actual overland flow path, roughness, slope, and temporal and spatial rainfall variations can overwhelm any assumptions about

physics. Equation (5-9) has the advantages of using the correct physical concept and resulting in a shorter t_c (and hence a more conservative design).

5. *Contributing Area*

The contributing area in the rational formula can be either the entire drainage area upstream of the point under study or some part of this area. The rational formula might yield a larger peak discharge for a subarea than for the entire drainage area if the subarea has either a larger runoff coefficient or a much shorter time of concentration than the total area. As a result, the rational formula should be applied in two ways: (1) using the entire drainage area, and (2) using only a portion of the most densely developed hydraulically connected area to see which portion of the tributary watershed governs.

Some fundamental points regarding the proper application of the rational method are illustrated in the following simple example.

Example 5-5: Use the rational formula to determine design discharges for the two inlets and two pipes shown in Figure 5.9. The design return period is 5 years. The local 5-year rainfall intensity duration relationship is fitted by the equation $i_e = 51.4 / (t_d + 7.8)^{0.75}$, where i_e is the average rainfall intensity in in./hr and t_d is the duration in minutes. Sub-catchment A has an area of 2.5 acres, 40 percent of which is impervious. Sub-catchment B has an area of 4.0 acres, 15 percent of which is impervious. The runoff coefficients are estimated to be 0.9 for the impervious surfaces and 0.2 for the pervious surfaces.

T_c is calculated by iterating between Equation 5-9 and the intensity-duration relationship above, and the results are shown in the following table:

			Catchment A		Catchment B	
T_d	i_e	$i_e^{0.4}$	$T_{c(imp)}$	$T_{c(per)}$	$T_{c(imp)}$	$T_{c(per)}$
5	7.59	2.24	9.95	17.23	9.95	25.36
10	5.93	2.03	10.98	19.01	10.98	27.99
15	4.92	1.89	11.79	20.42	11.79	30.06
20	4.24	1.78	12.52	21.69	12.52	31.92
25	3.75	1.69	—	22.84	13.18	34.68
30	3.37	1.62	—	—	—	35.07
35	3.07	1.56	—	—	—	36.42
40	2.82	1.51	—	—	—	37.62

We can see that, for Catchment A, the T_c is between 10–15 minutes for the impervious area, and 20–25 minutes for the pervious area. Similarly, for Catchment B, T_c is between 10–15 minutes (impervious) and 35 and 40 minutes (pervious). Interpolating, we determine that:

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	Catchment A		Catchment B	
	Imp	Per	Imp	Per
T_c	11	22	11	37
i_e	5.69	4.03	5.69	2.96

(a) Inlet 1 and Pipe 1—Considering the entire Catchment A, the contributing area is 2.5 acres, the time of concentration is 22 minutes, the corresponding average rainfall intensity is 4.03 in./hr, and the area-weighted average runoff coefficient is 0.48. The rational formula yields a peak discharge of 4.89 cfs for this case.

Considering only the impervious portion of the catchment, the contributing area is 1.0 acres, the time of concentration is 11 minutes, the

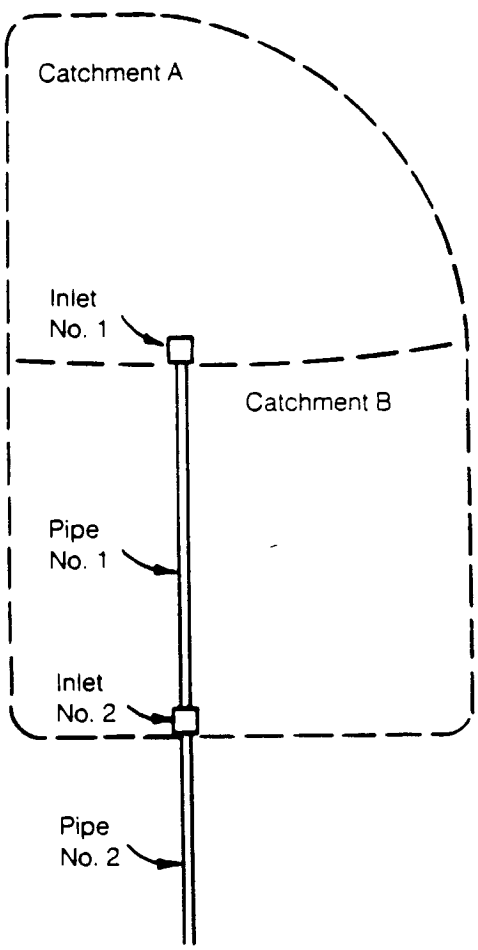

Figure 5.9—Example for rational method. For catchment A: A = 2.5 ac, I = 40%, C = 0.9 (impervious), C = 0.2 (pervious), C = 0.48 (area-weighted), L (impervious) = 200 ft, L (pervious) = 250 ft, S = 0.01, n (impervious) = 0.1, n (pervious) = 0.2. For catchment B: A = 4.0 ac, I = 15%, C = 0.9 (impervious), C = 0.2 (pervious), C = 0.30 (area-weighted), L (impervious) = 200 ft, L (pervious) = 475 ft, S = 0.01, n (impervious) = 0.1, n (pervious) = 0.2.

corresponding average rainfall intensity is 5.69 in/hr, and the runoff coefficient is 0.9. The rational formula yields a peak discharge of 5.1 cfs for this case. Therefore the design discharge is 5.1 cfs, controlled by the runoff from the impervious surfaces.

(b) Inlet 2—Considering the entire Catchment B, the contributing area is 4.0 acres, the time of concentration is 37 minutes, the corresponding average rainfall intensity is 2.96 in./hr, and the area-weighted average runoff coefficient is 0.30. The rational formula yields a peak discharge of 3.6 cfs for this case.

Considering only the impervious portion of the catchment, the contributing area is 0.6 acres, the time of concentration is 11 minutes, the corresponding average rainfall intensity is 5.69 in/hr, and the runoff coefficient is 0.9. The rational formula yields a peak discharge of 3.1 cfs for this case. Therefore the design discharge is 3.6 cfs, controlled by the runoff from the entire basin.

(c) Pipe 2—Consider first the case where the entire 6.5-acre area tributary to Pipe 2 is contributing runoff. To determine the time of concentration for this area, one would first compute the travel time in Pipe 1 using a hydraulic formula. Assume here that this travel time is 2 minutes. The travel time from the most remote point in Catchment A is 13 minutes, the sum of the inlet time for the pervious portion of this catchment and the travel time in Pipe 1. The travel time from the most remote point in Catchment B is 37 minutes, the inlet time for the entire catchment. The time of concentration for the entire 6.5-acre area is 37 minutes, the longest travel time to the point under study. The corresponding rainfall intensity is 2.96 in./hr, and the area-weighted average runoff coefficient is 0.37. The rational formula yields a peak discharge of 7.11 cfs for this case.

Next consider the case where only the impervious portions of Catchments A and B are contributing runoff. The contributing area is 1.6 acres. The time of concentration is 13 minutes, the sum of the inlet time for the impervious portion of Catchment A and the travel time in Pipe 1. The corresponding average rainfall intensity is 5.28 in./hr, and the runoff coefficient is 0.9. The rational formula yields a peak discharge of 7.6 cfs for this case. Therefore the design discharge is 7.6 cfs, controlled by the runoff from the impervious area.

H. Assessing the Reasonableness of Computed Flows

After computing hydrographs or peak discharges for the desired return periods, the engineer must assess the reasonableness of the results. This assessment is generally based on the engineer's own experience and the experience of other engineers in previous flood studies on hydrologically similar watersheds. Results for equal return periods on watersheds with similar physical characteristics in the same vicinity can be compared in terms of runoff rates per unit surface area. For example, in the Denver area, engineers assume that the 100-year runoff rate is about 1 cfs per acre for undeveloped land whenever the watershed is approximately one square mile in size. Any result that de-

viated significantly (i.e., more than minus 25% or plus 50%) from this value would merit reappraisal. Approximate unit rates of runoff and flood envelope curves may be available from an agency of local or state government. In comparing unit runoff rates for catchments of different sizes, one should remember that the peak discharge per unit surface area tends to decrease as catchment area increases because of storage effects and spatial variability of rainfall.

The reasonableness of a flow estimate can also be checked by comparison with the results of another, usually more approximate, hydrologic procedure. Peak discharges may differ significantly (a variation of 25% or more is common), but other hydraulic characteristics such as the total volume and time to peak should be comparable. Substantial differences may indicate an error in applying one of the procedures.

If hydrographs or peak discharges are generated by more than one method, the engineer must decide which results to use as a basis for design. This decision requires thoughtful appraisal, and the engineer should have a sound basis for his ultimate decision. Each procedure should be carefully considered with regard to its origin, assumptions, limitations, applicability to local conditions, and the reliability of the needed input data. If, based on these considerations, one procedure appears to be more appropriate than the others, the results of this procedure should be used as the basis for design. If no one procedure is clearly most appropriate, one could reasonably opt for the results of the procedure most often used by local engineers. If one procedure produces flows that are significantly lower than alternative procedures, these flows should not be used as a basis for design unless this decision can be justified convincingly.

I. Interpretation of Computed Flows

After hydrographs or peak discharges have been computed for various return periods, the engineer must select the design return periods for the minor (primary) and major drainage systems. The local government usually specifies the minimum return period to be used in designing the minor system. However, it may be desirable to examine the additional cost of designing for a higher return period. In some instances the frequency with which the capacity of the minor system is exceeded can be reduced greatly for a rather nominal increase in cost. In designing the major drainage system, the engineer should keep in mind that the major system design runoff event probably does not represent the worst possible case. Where a potential for catastrophic losses exists, one should attempt to evaluate the consequences of larger flood.

J. Significance of Historic Flood Data

The engineer should determine whether any information on historic floods is available for the catchment under study or for any hydrologically similar catchment nearby. Information on urban floods can often be obtained from local offices of the U.S. Geological Survey (Water

Resources Division), or by interviewing long-time residents of the community. Municipal public works personnel also may be able to provide useful information. If the approximate date of a historic flood is known, a newspaper account often can be found in the local public library. Rainfall data should be sought in the records of the National Weather Service. Peak discharges can often be estimated roughly from high-water marks.

It is usually not possible to make a reliable estimate of the return period of a historic flood on an ungauged stream. Therefore historic flood data are generally of limited value in assessing the reasonableness of computed flows. However, if such data are available, the engineer should attempt to determine how the storm drainage system would perform under these extreme conditions. The results of such an analysis may suggest ways in which the major drainage system may be modified to improve public safety during extreme flooding.

III. QUALITY OF STORM SEWER DISCHARGES

Sources of pollutants that affect the quality of discharges from separate storm sewer systems can be described in terms of two major classes: non-storm water sources; and runoff related sources. This section describes these two classes of pollutant sources as well as water quality impacts associated with discharges from separate storm sewers. Discharges from combined sewer overflows can differ significantly, and are discussed in more detail in other literature.

A. Pollutant Sources: Non-Storm Water Sources

Although separate storm sewers are primarily designed to remove runoff from storm events, materials other than storm water find their way into, and are ultimately discharged from, separate storm sewers. A wide range of pollutants can be associated with non-storm water discharges, including pathogens, metals, nutrients, oil and grease, metals, phenols, and solvents. Removal of non-storm water sources of pollutants often provides opportunities for dramatic improvement in the quality of discharges from separate storm sewers.

Major classes of non-storm water discharges to storm sewers include:

1. Illicit or Cross Connections

Illicit connections, also referred to as cross connections, to separate storm water sewers are physically connected conveyances that carry untreated wastewaters other than storm water. Illicit connections take a variety of forms, including improper connections of residential sewer service lines or sumps, cross-connections with sanitary sewers, improper connections of industrial sewer lines, and the improper disposal of wastes to floor drains or outdoor drains connected to separate storm sewers. For many of these connections, there is a mistaken belief that

materials are going to a sanitary sewer or some other type of treatment facility.

Illicit connections to separate storm sewers can occur in a number of ways. In older sections of cities with separate storm sewers, illicit connections can often be traced to the initial development of the storm sewer system. Illicit connections can occur in newer systems where flows in sanitary sewers have grown to exceed the hydraulic capacity of sanitary sewers, and formal connections or overflow devices have been installed or where holes are punched into the sanitary sewer to relieve the sanitary sewer of high flows. Discharges from malfunctioning sanitary sewage pumping stations can also be directed towards storm sewers. Incomplete separation of combined sewers can result in significant numbers of cross-connections between the sanitary sewer system and the storm sewer system.

2. *Interactions with Sewage Systems*

Sewage exfiltration out of a sanitary sewer collection system can result from aged sanitary sewers, poorly constructed manholes and joints, and main breaks. Sewage from a leaky sanitary system can flow to a storm sewer or contaminate ground water supplies.

3. *Improper Disposal*

Discharges from separate storm sewers can be contaminated by materials that are improperly disposed directly to a catchbasin or are improperly disposed to the ground and drained or washed into a storm sewer. These materials include used oil, household toxic materials, radiator fluids, litter and cement-truck washings. Direct improper disposal to separate storm sewers occurs in part because much of the public believes that disposal of materials to street catchbasins is an environmentally sound practice, rather than a conduit to a receiving water. In some areas, the use of used oil for road oiling is an accepted practice.

4. *Spills*

Spilled materials often have a high potential for entering man-made drainage systems. A wide variety of materials may spill during transportation, transfer, use, and storage.

5. *Malfunctioning Septic Tanks*

In rural and suburban areas served by septic tanks, malfunctioning septic systems can contribute pollutants to separate storm sewers. Surface malfunctions are caused by clogged or impermeable soils, or when stopped up or collapsed pipes force untreated wastewater to the surface. These discharges have high bacteria, nitrate, and nutrient levels and can contain a variety of household chemicals. One type of improper

remedy to a surface malfunction is to install a pipe or trench over soil absorption systems to route overflow away from the septic system, resulting in direct discharges to drainage ditches, empty lots, or surface waters.

6. Infiltration of Contaminated Ground Water

Many separate storm sewers are subject to infiltration from surrounding groundwater. Usually this infiltrated water is not contaminated and poses no direct pollutant threat to the receiving water. However, separate storm sewers may serve as a conduit to surface waters for ground water contaminated by industrial or other sources.

B. Pollutant Sources: Runoff Related Sources

The type and concentration of pollutants in runoff discharged from separate storm sewer systems will depend on the types of land use activities occurring in the area served by the storm sewer system. The quality of runoff from several major types of land uses are discussed below:

1. Runoff from Residential and Commercial Areas

Pollutants in runoff from residential and commercial lands has been addressed in a number of studies, including the Nationwide Urban Runoff Program (NURP). The NURP program evaluated data collected between 1978 and 1983 from 81 sites in 22 of the 28 cities funded by the program. Of the 81 sites selected, 39 were completely or primarily residential, 14 were commercial, 20 were mixed commercial and residential, and 8 were runoff from open space in urban areas. The NURP study provides insight on what can be considered background levels of pollutants in runoff from residential and commercial land uses as sites were carefully selected so they were not impacted by pollutant contributions from construction sites, industrial activities, or illicit connections. Data from several sites had to be eliminated from the study because of elevated pollutant loads associated with these sources.

The majority of samples collected in the NURP study were analyzed for seven conventional pollutants and three metals. A summary of the concentrations of these parameters is provided in Table 5.7.

One way to evaluate the NURP data is to compare annual pollutant loads in runoff from commercial and residential areas with discharges from sewage treatment plants providing secondary treatment. Such a comparison indicates that on an annual loading basis: suspended solids in runoff are around an order of magnitude or more greater; chemical oxygen demand (COD) in runoff is comparable in magnitude; and nutrients in runoff are around an order of magnitude less than annual loadings of discharges from sewage treatment plants providing secondary treatment. When analyzing annual pollutant loadings, it is im-

TABLE 5.7: Water Quality Characteristics of Runoff from Residential and Commercial Areas

Constituent (1)	Average Residential or Commercial Site Concentration (2)	Weighted Mean Residential or Commercial Site Concentration (3)	NURP Recommendations for Load Estimates (4)
TSS	239 mg/l	180 mg/l	180–548 mg/l
BOD	12 mg/l	12 mg/l	12–19 mg/l
COD	94 mg/l	82 mg/l	82–178 mg/l
Total P	0.5 mg/l	0.42 mg/l	0.42–0.88 mg/l
Sol. P	0.15 mg/l	0.15 mg/l	0.15–0.28 mg/l
TKN	2.3 mg/l	1.9 mg/l	1.90–4.18 mg/l
NO2 + 3 − N	1.4 mg/l	0.86 mg/l	0.86–2.2 mg/l
Total Cu	53 ug/l	43 ug/l	43–118 ug/l
Total Pb	238 ug/l	182 ug/l	182–443 ug/l
Total Zn	353 ug/l	202 ug/l	202–633 ug/l

Developed from results of the nationwide urban runoff program (EPA 1983).

portant to recognize that discharges of runoff are highly intermittent, and short-term loadings associated with individual events could be high and may have shock loading effects on receiving waters.

Additionally, 121 samples at 61 sites were analyzed for 120 of the pollutants EPA classifies as priority pollutants. Heavy metals were by far the most prevalent priority pollutant found in the study, with 10 metals detected in discharges at frequencies of greater than 10%, and copper, lead, and zinc each found in at least 91% of the samples. Sixty-three of the 106 organics measured were detected, with concentrations of organic pollutants in discharges exceeding water quality criteria less frequently than with heavy metals. The NURP found fecal coliforms in runoff from residential and commercial lands at concentrations approaching dilute sewage at a number of sites. Seventeen sites analyzed fecal coliform levels, with fecal coliform counts in runoff typically in the tens of thousands per 100 ml during warm weather conditions (average NURP site concentrations were 27,000 counts/100 ml), with lower concentrations during colder weather (average NURP site values were 1,000 counts/100 ml for cold weather). Other pollutants were either not considered in the NURP study (e.g. oil and grease, floatables, chlorides, non-polar pesticides, asbestos, etc.) or were found less frequently.

When considering relatively large commercial and residential drainage basins, the most important factors influencing pollutant loadings are usually the degree of imperviousness and the amount of precipitation. The NURP concluded that, for general planning purposes, the concentrations of pollutants in runoff from different large residential

and commercial areas can be assumed to be roughly equivalent, but the degree of imperviousness plays an important role in determining pollutant loads. Central business districts, which have a very high degree of imperviousness, will usually have the highest pollutant loadings per unit area. Commercial land uses can also have high degrees of imperviousness. The degree of imperviousness of residential lands is generally significantly lower than commercial land uses, and depends on the type of housing provided and the resulting density.

Seasonal variations are also important considerations in many parts of the country. The Milwaukee NURP project showed that for northern climates, concentrations of total suspended solids and associated metals were highest in winter, with spring pollutant concentrations following closely. High winter concentrations were attributed to the release of pollutants accumulated in snow. In addition, chloride concentrations will be highest in the winter in areas were road salting is performed. In areas with long dry seasons, accumulated pollutants result in peak levels of heavy metal and oil and grease in runoff from early rain season storm events. Peak concentration levels can be on the order of three or four times greater than the concentrations in discharges occurring later in the season. Areas with high intensity rainfalls, such as the southwest and parts of Texas, can have high rates of erosion from unprotected soils.

2. *Runoff from Construction Sites*

The amount of sediment in storm water discharges from construction sites can vary considerably, depending on whether effective management practices have been implemented at the site. Uncontrolled or inadequately controlled construction site sediment loads have been reported to be on the order of 35–45 tons/acre/year. Sediment runoff rates from construction sites are typically 10–20 times those from agricultural lands, and typically 1,000–2,000 times that of forest lands. Over a short period of time, construction sites can contribute more sediment to streams than was previously deposited over several decades.

3. *Runoff from Industrial Lands*

Discharges from separate storm sewers serving industrial lands may contain a larger number of toxic constitutes at higher concentrations. In general, a greater variety and larger amounts of toxic materials are used, produced, stored or transported in industrial areas. Material management practices, and atmospheric deposition can contribute to significant levels of toxic constituents in runoff. Many industrial areas have a high potential for illicit connections, spills, leaks, and other sources that may contribute a wide variety of pollutants to discharges from separate storm sewer systems. In addition, many heavy industrial areas are highly impervious, which results in high volumes of runoff with high pollutant loads.

4. *Runoff from Roads and Highways*

Pollutant concentrations in runoff from roads and highways are generally higher than those found in typical runoff from residential and commercial areas. Traffic-related pollutants come from leakage of oil, fuel oil, hydraulic fluids, coolants, incomplete combustion of fuel, clutch and brake lining wear, particulate exhaust emissions and debris from vehicles. Rust, dirt, metals, litter, plastic, and glass are pollutants from weathering and wear.

Research indicates that the median concentrations of pollutants in urban road and highway runoff are typically three times higher than pollutant concentrations in runoff from roads in rural areas. Higher pollutant concentrations in urban areas were attributed to higher traffic volumes and more atmospheric deposition of pollutants.

Road maintenance activities, including right-of-way grass mowing, vegetation control, road repair, snow removal, and road de-icing activities, can significantly impact the pollutants in runoff. Spraying of herbicides and growth regulators has become an increasingly popular method to control vegetation along roadsides. De-icing salts can be major sources of sodium and chlorides in storm water runoff from roads and highways. De-icing salts can also be a source of toxic metals (i.e., lead, nickel, chromium) and cyanide (used as an anticaking agent to prevent the granulated salt from solidifying).

5. *Estimation of Pollutant Loads*

A number of models have been developed for estimating pollutant loads in discharges from separate storm sewers. A detailed description of quality and quantity models is provided in Chapter 7 of this manual. As discussed previously, pollutants in discharges from separate storm sewers can come from a variety of sources, making pollutant loading estimates without discharge-specific pollutant concentration data difficult. Typically, where discharge-specific data is not available, models assume pollutants concentrations in discharges can be approximated by the pollutant concentrations associated with runoff from lands used for residential and commercial activities which are the predominate land uses in most urbanized areas, typically occupying between 55 to 85 percent of the total area. Such an assumption can be used to provide initial planning estimates of the background level of pollutants expected in discharges from separate storm sewers. However, the resulting estimates should be presented with the caveats that they do not account for pollutants from many sources such as industrial runoff, illicit connections, construction site runoff, improper dumping, spills, etc, and that pollutants in separate storm sewer discharges exhibit considerable variability both from event to event and from place to place. Bearing this in mind, two relatively simple methods that may be appropriate for providing initial planning level estimates of pollutants are discussed below:

Concentration Times Flow Pollutant loads associated with discharges from separate storm sewer systems can be estimated by multiplying the estimated volume of the discharge over a given time period (e.g. event or annual) by an estimated representative concentration for the event(s) to be modelled. Hydraulic models that can be used to estimate discharge volumes are discussed in more detail in Chapter 7 of this manual. Table 5.7 summarized NURP data of the event mean concentrations for 10 pollutants in runoff from residential and commercial land uses.

Regression Formulas The United States Geological Survey (USGS) (Driver and Tasker, 1988) has developed regression equations for estimating pollutant loads in urban runoff. The regression equations are based on the NURP data base supplemented by USGS data for 2813 storms at 173 urban stations in 30 metropolitan areas throughout the United States. Equations are provided for estimating pollutant loads from individual storms, and mean seasonal or mean annual pollutant loads for eleven constituents: chemical oxygen demand, suspended solids, dissolved solids, total nitrogen, total Kjeldahl nitrogen, total phosphorus, dissolved phosphorus, cadmium, copper, lead, and zinc.

C. Water Quality Impacts of Storm Water Discharges

A National Water Quality Inventory, (EPA 1988) indicates that discharges from municipal separate storm sewers are a major source of surface water use impairment. The Inventory, which is based on reports from the States, indicates that of the rivers, lakes, and estuaries that were assessed by States, roughly 70–75% are supporting the uses for which they are designated. Of those waters that are experiencing use impairment, separate storm sewers are the leading cause of impairment for 7% of rivers and streams, 41% of estuaries, 8% of lakes, 20% of coastal areas and 35% of the Great Lakes shoreline. The National Water Quality Inventory generally only addresses larger receiving water bodies and does not address major portions of the natural drainage system of most watersheds, such as smaller feeder streams and wetlands where impacts from separate storm sewers are often the most dramatic. A number of other major sources cited in the report can, to varying degrees, discharge through urban drainage systems including construction, land disposal of wastes, and resource extraction. With continued urban expansion, further degradation of surface waters from separate storm sewers remains a concern.

The effects of discharges from separate storm sewers on receiving waters depend on the hydrology and chemistry of the receiving body as well as the quantity and quality of the discharge. This manual will describe classes of general receiving water impacts that can occur; however, local evaluations such as instream biosurveys, sampling of sediment quality, and measuring discharge quantity and quality, can provide the most accurate assessment of impacts to a given receiving water.

1. *Parameters Associated with Impacts*

Siltation/Sedimentation Siltation from sediment pollutant loads can cause a broad range of interrelated impacts in receiving waters including:

(a) Loss of benthic habitat—Channel scour and bank erosion can result in habitat destruction. Suspended solids may be deposited as sediment bars or sediment blankets in pools and other areas and may smother benthic organisms, including the eggs and immature forms of free-swimming organisms.
(b) Reduced water storage capacity—Increased sediment loads reduce water storage capacity in reservoirs (in urban areas, these loads result largely from construction activities). Sediment loads can also reduce stream depths, decreasing their retention and conveyance capacity, which can then result in increased flooding.
(c) Impaired oxygen exchange—Increased turbidity levels may impair the ability of aquatic organisms to obtain dissolved oxygen from the water by interfering with the gill movements and associated water circulation.
(d) Decreased light penetration—The depth of light penetration into surface waters is sharply diminished by turbidity. As a result, photosynthetic activity and food sources are reduced. Loss of submerged aquatic vegetation may also remove habitat for juvenile fish and shellfish.
(e) Increased water treatment costs—Sediments can increase the costs of treating potable water supplies. Inadequate sediment removal may limit the effectiveness of chlorination.

In addition, many of the pollutants associated with runoff may become chemically or physically bound with sediment particles and accumulate in the bottom of receiving waters. As a result, many of the pollutants in urban runoff are trapped in bottom sediments and are thereby immobilized. It is possible that these accumulated pollutants may remobilize and contaminate the overlying water column and benthic biota, and may even enter the food chain. However, much research is still needed to define the physiochemical and biological processes and activities that may be related to these pollutants. Oxygen-demanding pollutants in sediment deposits can create oxygen deficits during and after storm water discharge events. Variable flows in receiving waters can resuspend sediments. The repetitive process of deposition, resuspension and redeposition of sediments can result in pollutants associated with sediments taking a long time to pass through receiving waters.

Organic Enrichment/Oxygen Demand Aquatic organisms such as fish and water-dwelling insects require minimum levels of dissolved oxygen (DO). Excessive oxygen—demanding pollutants can lead to periods of oxygen sag which may cause fish kills or create anoxic conditions accompanied by foul-smelling odors. Oxygen levels in receiving waters can be lowered by the decomposition of organic matter by microorganisms, by the chemical oxidation of material, or by aquatic vegetation that uses more oxygen at night than it produces.

The two parameters most commonly used to describe the oxygen demand of pollutants are the five-day biological oxygen demand (BOD5) and chemical oxygen demand (COD). Of the two, COD is more accurate for the purpose of comparing the oxygen demand of storm water discharges to the oxygen demand of other types of discharges. The BOD5 test underestimates the true oxygen demand of storm water because heavy metals in the storm water slow the bacterial action used in the test.

The impacts of oxygen-demanding pollutants can be more dramatic in shallow, slow-moving waters due to limited aeration and the tendency of these pollutants to accumulate in bottom sediments.

Pathogens Pathogens are disease-causing organisms including viruses and some bacteria. Water-borne pathogens can be transmitted to man or animals through direct contact recreation, drinking water supplies or through eating contaminated shellfish. Pathogens may enter separate storm sewers from leaking sanitary sewers, cross-connections with sanitary sewers, malfunctioning septic tanks and animal wastes.

Toxicity (Metals, Toxic Organics, Pesticides, Inorganics, and Oil and Grease) A wide range of chemicals can exhibit toxicity, including metals, organics, pesticides, inorganic pollutants, and oil and grease. Toxic impacts can be classified in terms of acute and chronic effects.

Many of the toxic metals and other toxic constituents in storm water discharges are attached to suspended solids in the discharge, and may accumulate in the bottom sediments of receiving waters where they may persist for long periods of time. Toxics concentrated in bottom sediments can cause adverse impacts on benthic organisms, may become resuspended during high flows resulting from other large storm events, or may dissolve into the water as parameters such as Ph and dissolved oxygen change. Accumulated pollutants in bottom sediments may also adversely affect fish during periods of continuous low flow.

Nutrients Excessive nutrients over-stimulate the growth of aquatic plants which may result in low oxygen levels, accelerated eutrophication, unsightly conditions, interference with navigation, interference with treatment processes, and disagreeable tastes and odors. Eutrophic conditions are evidenced by surface algal scums, reduced water clarity, odors, and dense algal growth on shallow water substrates. Algal blooms block light from submerged aquatic vegetation which may remove habitat for juvenile fish and shellfish. After blooms or at the end of a growing season, the decomposition of dead vegetation may result in reduced oxygen levels, leading to fish kills and mass mortality of benthic organisms.

Excessive nutrients usually have more adverse affects in surface water bodies with slow flushing rates, such as slow moving rivers, lakes and estuaries. Nutrients delivered during storm events may settle out, and later be solubilized or resuspended.

Temperature Increased temperature can have detrimental effects on fish and other aquatic life during various stages of their life cycle. Water holds less oxygen as it gets warmer, which may affect habitat and make the water more susceptible to oxygen-demanding pollutants.

Floatables, Plastics Large amounts of litter and plastics are flushed from storm sewers. Litter and other floatables can degrade aesthetic quality and diminish the attractiveness of receiving waters and lower the value of adjacent property. Litter can impact the operating effectiveness of the drainage system and related facilities such as detention ponds. Economic losses caused by the aesthetic degradation of recreational areas such as beaches have been found to be significant. Plastic debris present hazards to wildlife.

2. *Assessing Impacts*

Assessing the impact (if any) of discharges from separate storm sewers on receiving waters is difficult for a number of reasons, including:

(a) Water quality impacts may be caused by a combination of the quality and quantity of discharges, or a combination of sources.
(b) Water quality impacts are site-specific.
(c) Water quality impacts may occur considerable distances downstream from discharge locations.
(d) Water quality standards may be difficult to relate to actual impacts.

Impacts: Quality and Quantity Impacts on receiving waters associated with storm water discharges can be discussed in terms of three general classes: 1) short-term changes in water quality; 2) long-term water quality impacts; and 3) physical impacts. Use impairment of receiving waters often is caused by a combination of all three types of impacts. Physical impacts and short-term water quality changes are generally more critical than long-term water quality impacts for receiving waters with relatively short residence times (such as smaller streams and rivers). Receiving waters with long residence times (lakes, estuaries) are generally more sensitive to long-term water quality changes, although certain physical changes, such as loss of reservoir capacity due to siltation, can be important.

Short-term changes in water quality occur during and shortly after storm events. Examples include periodic dissolved oxygen depressions due to oxidation of pollutants, short-term increases in the receiving water concentrations of one or more toxic pollutants, high bacteria levels, and peak acidity. These conditions can result in fish kills, loss of submerged macrophytes, and other temporary impairment of uses.

Long-term water quality impacts are caused by the cumulative effects associated with repeated storm water discharges. These impacts often result from pollutants from a number of different types of sources. Examples of the long-term water quality impacts that runoff can cause

or contribute to include: depressed dissolved oxygen caused by the oxygen demanding pollutants in bottom sediments; biological accumulation of toxics as a result of uptake by organisms in the food chain; chronic toxicity to organisms subject to repeated exposures of toxic pollutants; destruction of benthic habitat; loss of storage capacity in receiving waters; and increased lake eutrophication. Long-term water quality impacts also include impacts caused by pollutants attached to suspended solids that settle in receiving waters and by nutrients that enter receiving water systems with long retention times. In both cases, water quality impacts are related to the increased residence times of pollutants in receiving waters. Long-term water quality impacts of pollutants from storm water discharges may be manifested during critical periods other than during storm events, such as during low stream flow conditions, or during sensitive life cycle stages of organisms. When evaluating long-term impacts, the cumulative and relative effects of seasonal and long-term pollutant loadings from all relevant sources (e.g., storm water, sewage treatment plants, industrial discharges, nonpoint sources, atmospheric deposition, in-place pollutants, etc.) should be considered.

Physical impacts due to erosional effects of high stream velocities that occur after the natural hydraulic cycle has been altered can deteriorate fish habitat. These changes are often accompanied by the installation of engineered structures such as concrete walls or underground culverts which may further degrade the habitat and aesthetic values of the receiving water. In addition, where ground water recharge has been limited by the placement of impervious structures on the land, dry weather base flows may be lowered to the detriment of the receiving water.

The Bellevue, Washington NURP project provides an example where water quality impacts were caused by a combination of factors. The project involved the study of a stream that received runoff from residential lands. The pollutant concentrations in the runoff to the stream had significantly lower concentrations than the average NURP site. Runoff to the stream did not appear to cause short term acute toxicity problems. However, massive fish kills were observed on several occasions during the study and were attributed to the illegal dumping of toxic materials into storm drains. In addition, the study recognized that potential long term problems may be associated with settleable solids, lead, and zinc which may have silted up spawning beds and introduced high concentrations of potentially toxic materials directly to the sediments. Up to three-fourths of the fish in the receiving stream had damaged gills and respiratory anomalies. Further, benthic organisms, which are sensitive indicators to environmental degradation, were rarely found in the receiving stream.

Site Specific Considerations Factors to consider when evaluating the pollutants in discharges from separate storm sewers include:

(a) Size of area served by separate storm sewers.
(b) Rainfall patterns and amounts and seasonal effects.
(c) Potential for non-storm water discharges to the separate storm sewer system.
(d) Receiving water characteristics.
(e) Interaction with other pollutant sources such as discharges from sewage treatment plants and combined sewer overflows.
(f) Potential for development and/or renovation.
(g) Potential for pollutants in storm water associated with industrial activity.

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Chapter 6
STORM DRAINAGE HYDRAULICS

I. INTRODUCTION

The purpose of this chapter is to discuss the basic hydraulic principles inherent in the design of a storm drainage system (Figure 6.1). The basic elements covered include street inlets, storm sewers, natural and man-made channels, culverts, detention basins, and outlet structures. The primary design objective for each of these elements is usually to provide a discharge capacity sufficient to convey the design flow at velocities that are self-cleansing without being destructive. In the case of storm sewers and man-made channels, the designer may be asked to make additional calculations involving hydraulic grade line or water surface profile, total energy grade line, normal and critical depth, and location of hydraulic jumps, surcharge, or out-of-bank flows.

The design and analysis of drainage system facilities proceed according to fundamental principles of water movement in open channels, closed conduits, and other special hydraulic structures. This chapter treats water as an incompressible Newtonian fluid, and emphasizes the following concepts and principles: flow classification; mass conservation; momentum conservation; total and specific energy; friction losses and minor losses; hydraulic jump; water surface profiles; special hydraulic structures; and flood routing by both hydraulic and hydrologic methods. Pump hydraulics are not discussed here because of the ample treatment provided in standard hydraulic texts.

II. FLOW CLASSIFICATION

Identifying the type of flow at the outset of a particular design problem is essential because the design equations themselves are often developed for specific flow classes. Flow in a storm drainage system can be classified in a number of ways, depending on the particular

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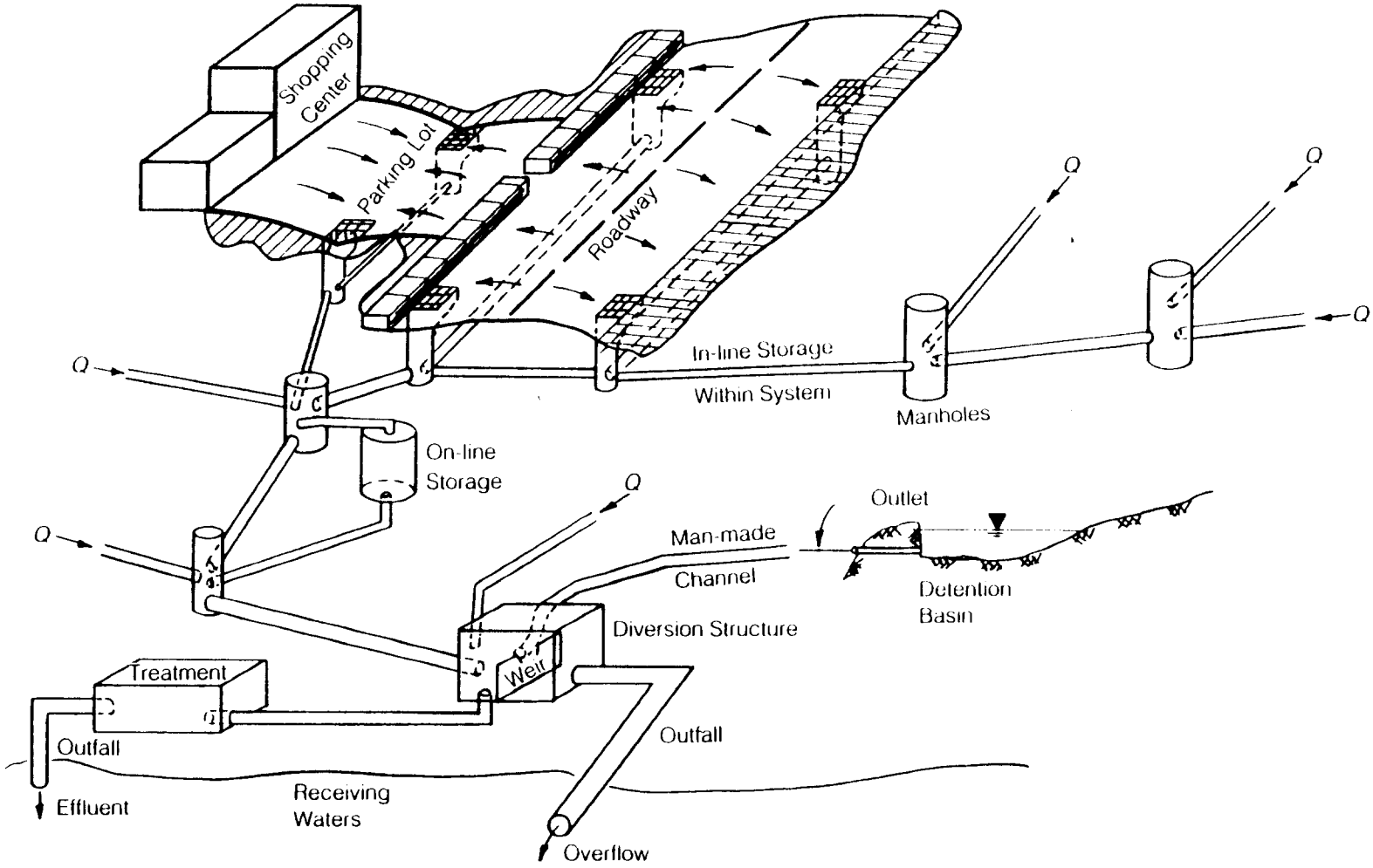

Figure 6.1—Principal hydraulic elements in urban storm drainage system.

circumstances. For example it might be classified as laminar vs. turbulent, steady vs. unsteady, uniform vs. nonuniform, gradually varied vs. rapidly varied, open channel vs. closed conduit, or free surface vs. pressure flow. Flow is thus classified according to certain properties that change with time and distance along the flow element. The brief outline below summarizes the traditional flow types encountered frequently in a storm sewer system:

A. Steady vs. Unsteady Flow

At a single location in a conduit or channel there may be changes in depth, velocity, or discharge that occur with time as the result of hydrograph inflows at an upstream inlet or perhaps tidal discharges at a lower location. Variations in depth, velocity, or discharge with time at a single point cause the flow to be unsteady. Steady flow on the other hand requires that discharge, depth, and velocity are constant with time, as in spring-fed baseflow in a small stream or interceptor. The flow is considered to be quasi-steady if depth, velocity, and discharge are changing very slowly over time, such as the sanitary flow in a combined sewer during dry weather. The flow in storm sewer systems will be unsteady during periods of rainfall-runoff. This unsteadiness is often ignored in the design of small municipal storm sewers and subdivision drainage systems based on peak flows only (note that it cannot be ignored in the design of complex networks involving looped sewers, diversion structures, and combined sewer overflow facilities).

B. Uniform vs. Nonuniform Flow

In uniform flow, depth, discharge, and velocity are constant with distance along the channel or conduit. Channel slope, energy or friction slope, and water surface slope are all equal. The depth of uniform flow is called the "normal" depth. Uniform flow can occur as non-pressure or free surface flow in a prismatic conduit flowing partially full. It also can occur in a storm sewer flowing full if minor losses due to contractions, expansions, and bends are negligible. Uniform flow equations such as the Manning or Hazen-Williams are still applicable, provided the slope of the energy grade line is used and the equations are applied in their proper flow ranges—Manning in the rough flow range and Hazen-Williams in the smooth flow range.

Nonuniform flow by contrast is characterized by changing depth and velocity with distance along the channel or sewer. Sometimes these depth and velocity changes take place over considerable lengths and the nonuniform flow is called gradually varied. Uniform flow equations are often applied to very short distance intervals within a gradually varied flow. Examples of gradually varied flow include sheet flow on paved surfaces, gutter flow, and flow in storm drains and floodways. Rapidly varied flow, on the other hand, produces abrupt changes in

depth and velocity over very short distances, as in the case of flow over an emergency spillway, through a hydraulic jump, or beneath a sluice gate. Rapidly varied flow usually involves wave phenomena which preclude the use of uniform flow formulas. Nonuniform flow can also be unsteady, as in the passage of a runoff peak or flood wave through a storm sewer or man-made channel and in fact most storm drainage flow can be classified as unsteady and nonuniform during periods of heavy runoff.

C. Open Channel vs. Closed Conduit Flow

Open channel flow has a free surface as in a natural stream, swale, or man-made channel. The slope of the channel will be classified as hydraulically mild, critical, or steep depending on whether this slope is less than, equal to, or greater than the critical slope computed for the channel on the basis of its critical depth, discharge, and roughness. The slope could of course also be horizontal or adverse. A closed conduit may flow full or partially full, depending on whether the runoff event is larger or smaller than the design value. Most storm sewers will be sized to flow full at the design discharge, although in certain instances where ground elevations are sufficient, a limited surcharge above the pipe crown may be permitted. Full flow in a conduit is confined without a free surface and is sometimes referred to as pipe flow or pressure flow. Gravity forces still govern, but the additional pressure head of any surcharge above the pipe crown must be taken into account. Closed conduits flowing partially full are analyzed as open channels. A procedure for computing normal depth and backwater profiles in partially full circular storm sewers has been described by Christensen (1984).

D. Laminar vs. Turbulent Flow

The flow in a pipe or channel may also be classified according to the nature of the flow streamlines and velocity distribution within the flow section, depending on whether viscous or inertia forces predominate. Laminar flow occurs in water supply conduits where viscous forces are predominant and the Reynolds number is less than about 500 (computed with hydraulic radius). Laminar flow may develop in overland sheet flow in the early stages of runoff on the rising side of the hydrograph well below the peak discharge. Storm sewer and open channel flow on the other hand tends to be turbulent, at least during periods of peak flow, and will have Reynolds numbers (computed from hydraulic radius) exceeding 500. Within the turbulent regime there are sub-classes designated as the smooth range, the transition range, and the rough range based on the thickness of the viscous sub-layer compared to the roughness size. These sub-classes are important since certain uniform flow formulas, such as the Manning equation, are considered more appropriate in the rough range, while the Hazen-Williams

and Colebrook-White equations are better suited for the smooth and transition ranges, respectively. In most design situations, the flow is turbulent, and the Manning equation can be used.

E. Subcritical vs. Supercritical Flow

The flow in open channels and closed conduits may also be classified according to the level of energy contained in the flow itself as represented by the Froude number. The subcritical range has Froude numbers less than 1.0 and is characterized by low velocities and high depths found typically on hydraulically mild slopes. Supercritical flow has a Froude number greater than 1.0 and is characterized by high velocities and low depths developed in a hydraulically steep channel or pipe. Critical flow occurs when the Froude number equals 1.0 and the actual depth is equal to the critical depth for the flow element. The classification of the flow according to subcritical, critical, or supercritical conditions is important for two reasons related to design of storm drainage facilities. First, the location of hydraulic jumps where the flow passes through an abrupt transition from supercritical to subcritical should always be determined by the designer so that the associated energy loss and depth increase can be accommodated in the designed system. Secondly, the location of a critical depth section in a channel or storm sewer is important because that section serves as a control from which water surface profile calculations can proceed. It also marks a point in the sewer or channel where a unique relation between depth and discharge exists and therefore constitutes an ideal flow monitoring location. More details about critical depth and water surface profile calculations are provided later in this chapter.

A systematic classification of flow types has been developed by Christensen (1985) and is shown in Figure 6.2. Having outlined all the principal flow types, it is fair to state that stormwater runoff usually will be classified as: unsteady, nonuniform, closed conduit flowing partially full, turbulent, and subcritical. Many departures from this general rule can of course occur. The analysis of complex unsteady, nonuniform flow problems sometimes can be accomplished satisfactorily by using less rigorous methods based on steady, uniform flow approximations. The purpose of the remaining sections in this chapter is to define fundamental flow relations and to provide examples illustrating the application of these principles to typical problems of storm drainage design and analysis.

III. CONSERVATION OF MASS

The principle of mass conservation states that the difference between the mass inflow and mass outflow rates over a time interval must equal the change in mass storage over the same time interval. Since stormwater is treated as incompressible, the mass rates are expressed as

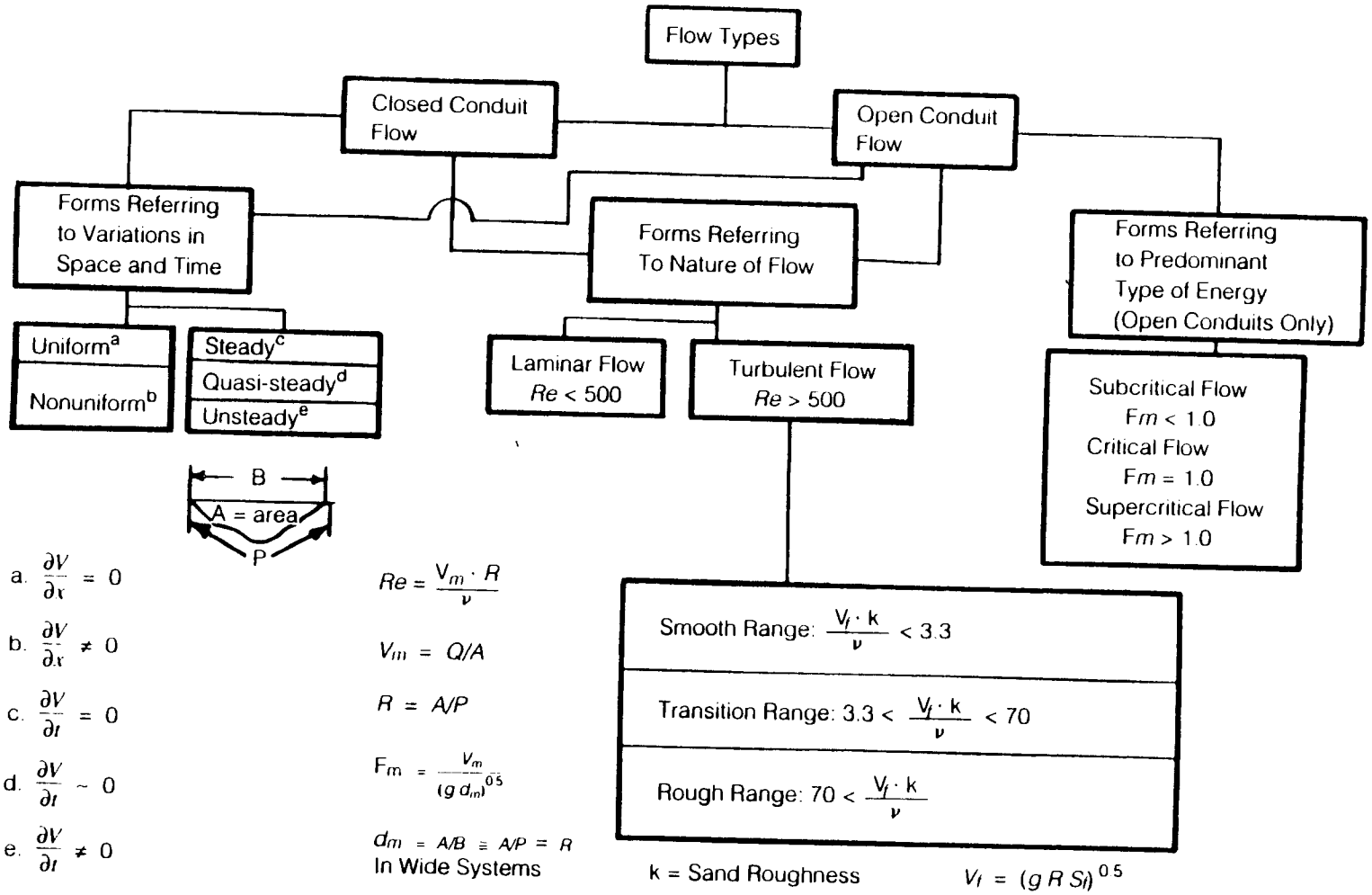

Figure 6.2—Schematic classification of flow types (B.A. Christensen, 1989).

volume rates over time. The general mass conservation equation, otherwise known as the continuity equation, is in any consistent unit system:

$$\frac{\Delta S}{\Delta t} = \Sigma \bar{I} - \Sigma \bar{O} \tag{6-1}$$

in which ΔS is storage change, Δt is the time interval, $\bar{I}$ is the average rate of inflow over the time interval, and $\bar{O}$ is the average outflow rate. For the system in Figure 6.3, the continuity equation written for the control volume represented by the manhole, would be:

$$\frac{\Delta S}{\Delta t} = \frac{A_s \times \Delta H}{t_1 - t_0} = Q_A + Q_U + Q_S + Q_D \tag{6-2}$$

where A_s is the cross-sectional area of the manhole and H is the depth change over the time interval $t_1 - {}_0$. If ΔS were zero, as it would be under steady flow, then the continuity equation for the system reduces to: $Q_D = Q_S + Q_A + Q_u$. Thus the continuity equation requires that the algebraic sum of inflows to a junction is equal to the sum of outflows from that junction when there is no storage change. Similarly, the continuity equation in pipe A assuming full flow is: $Q_A = A_1V_1 = A_2V_2$, where the subscripts 1 and 2 refer to two sections normal to the flow, A is the cross-sectional area, and V is mean velocity in the cross-section. Again, Figure 6.3 illustrates this situation.

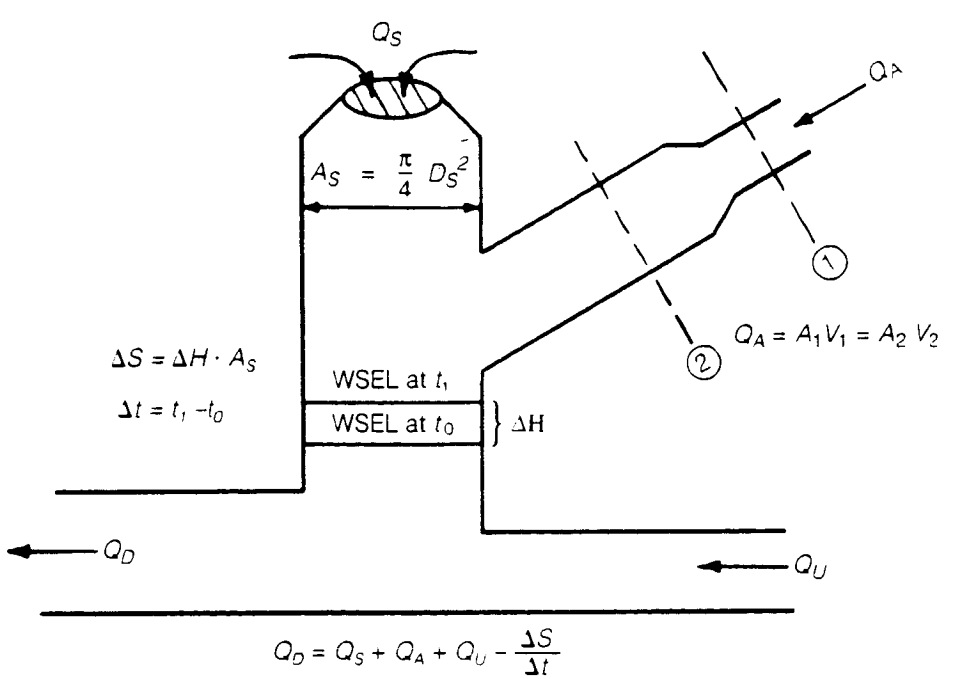

Figure 6.3—Continuity principle applied to unsteady and nonuniform flow: Q_S = street inlet inflow; Q_A = entering flow pipe A; Q_U = entering flow pipe U; and Q_D = exiting pipe flow.

During flood conditions in an open channel or storm sewer, flow is both unsteady and nonuniform, and the discharge into any control volume will not equal the discharge out of the control volume. If Q is replaced with its equivalent, AV, and the time change of storage within the control volume is taken into account, the partial differential form of the equation of continuity becomes (without loss or gain of water along the length of the control volume):

$$V\frac{\partial A}{\partial X} + A\frac{\partial V}{\partial X} + b\frac{\partial y}{\partial t} = 0 \qquad (6\text{-}3)$$

in which B is the water surface width, Y is the depth of water, X is in the direction of flow, and t is time. Figure 6.4 gives a definition sketch for the control volume under nonuniform unsteady flow in an open channel.

Users of computer models that simulate the routing of stormwater flows through conduits must be careful to confirm that continuity is being maintained by the routing procedure. Approximations have to be made in any dynamic routing algorithm, and under extreme conditions the model may over or underestimate routed flow. Depending on computational structure, some models may be prone to numerical

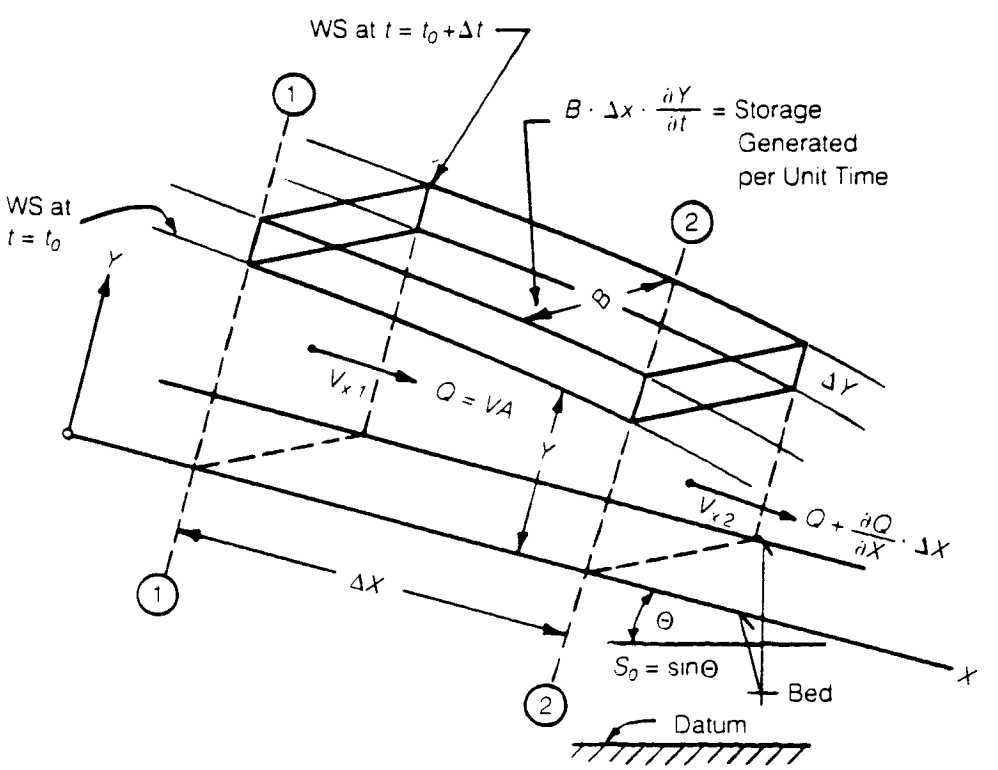

Figure 6.4—Definition sketch for nonuniform and unsteady flow in an open channel (See equation 6.3).

truncation or round off error. A check of cumulative inflow volume versus cumulative outflow volume will provide a rough check for the presence of continuity errors and indicate the significance of numerical truncation errors.

IV. CONSERVATION OF MOMENTUM

The momentum conservation equation is derived from Newton's second law which states that the time rate of change in linear momentum of a fluid mass equals the sum of external forces acting on that mass. For steady, uniform flow there are no velocity changes with time or distance and hence the net sum of external forces must be zero. For the nonuniform flow shown in Figure 6.4 the momentum conservation principle requires that the sum of external forces acting on the control volume in the X-direction between sections 1 and 2 be equal to the momentum flux or time rate of change in linear momentum, also in the X-direction. For the channel shown in Figure 6.4, the forces in the X-direction include gravity, friction, and hydrostatic pressure imbalance. Other forces due to contraction/expansion, wind shear, and rainfall momentum transfer can also be included depending on the particular application. For the case of steady, nonuniform flow in the open channel element of Figure 6.4, the momentum equation is shown in (6-4):

$$\Sigma F_x = \frac{\gamma Q}{g}(V_{x2} - V_{x1}) \tag{6-4}$$

in which ΣF_x is the summation of the X components of the external forces acting on the fluid body, γ is the specific weight, Q is the discharge, V_x is the spatial mean velocity in the x-direction at cross-sections 1 and 2, and g is the acceleration due to gravity.

This form of the momentum equation is adequate for steady flow in storm sewers and man-made channels which have simple prismatic cross-sections. The assumption of the velocity being approximately equal to the mean velocity across the channel is usually sufficiently accurate. However, in natural channels with flood plains and in complex man-made channels, this assumption can be inaccurate. In these cases, a momentum correction factor, β, must be applied to each of the velocity terms in Equation (6-4). The equation is then written as:

$$\Sigma F_x = \frac{\gamma Q}{g}(\beta_2 V_{x2} - \beta_1 V_{x1}) \tag{6-5}$$

where

$$\beta = \frac{\int V^2 dA}{V_m^2 A}$$

Under actual rainfall-runoff conditions, the flow in man-made channels and storm drains is unsteady and nonuniform. Referring to Figure 6.4, the force balance equation becomes:

$$\Sigma F_x = \gamma A S_o X - A S_f \Delta X - A \left(\frac{\partial Y}{\partial X}\right) \Delta X \qquad \text{(6-5a)}$$

where S_o is the channel or pipe slope defined as $\sin\theta$, and S_f is the energy or friction slope. The terms on the right hand side of equation (6-5a) represent weight, friction, and hydrostatic pressure forces, respectively acting in the X-direction. Letting $\Delta X -> \partial X$, $\beta = 1.0$, and setting ΣF_x in equation (6-5a) equal to the rate of change of momentum in the main fluid mass entering from upstream in the element shown in Figure 6.4, the momentum equation can be written in simplified form:

$$\frac{\partial V}{\partial t} + V\left(\frac{\partial V}{\partial X}\right) + g\left(\frac{\partial Y}{\partial X}\right) = g(S_o - S_f) \qquad \text{(6-5b)}$$

where forces due to local losses, wind shear, and lateral inflows have been neglected. Equations (6-3) and (6-5b), together, form the Saint-Venant or gradually varied unsteady flow equations, after Barre de Saint-Venant who first developed them for unsteady flow in an open channel in 1871. It should be noted that equation 6-5b can be rewritten in terms of the major flow classes discussed earlier in this chapter as (Henderson 1966):

$$S_f = S_o - \left(\frac{\partial Y}{\partial X}\right) - \frac{V}{g}\left(\frac{\partial V}{\partial X}\right) - \frac{1}{g}\left(\frac{\partial V}{\partial t}\right) \qquad \text{(6-5c)}$$

steady uniform flow

steady nonuniform flow

unsteady nonuniform flow

Equations (6-3) and (6-5b) are called the nonconservation form of the gradually varied unsteady flow equations because they are written in terms of velocity, V. For reasons of numerical stability, the conservation form, written in terms of discharge, Q, is often used (as in the extended transport hydraulic routing routine (EXTRAN) in the EPA SWMM program). Numerical solution of the full dynamic or gradually varied unsteady flow equation is important when analyzing complex sewer networks where pipe looping and various diversion/outfall structures can create significant backwater effects. These act to reduce the hydraulic

capacity of sewer elements leading to increased flooding and surcharge in heavily developed urban watersheds.

The solution of the full equations (6-3) and (6-5b) and the kinematic approximation for sewer and channel routing is discussed in a later section of this chapter. It is important to note that equation (6-5b) can also be developed from energy conservation principles, as well as from Newton's second law as shown above. The energy equation is introduced below because of its importance in developing the energy grade line and water surface profiles in steady flow sewer/channel problems.

V. CONSERVATION OF ENERGY

The total energy in a moving fluid at any point is the sum of potential energy, pressure energy, and kinetic energy. The energy conservation principle states that, in an ideal fluid without external energy sources and sinks, this total energy sum does not change along the flow element, but that only its distribution among the individual energy components changes. Thus, total fluid energy, although transformed, is always conserved. In the field of applied hydraulics, we must account not only for these principal energy components, but also for other energy gains and losses by boundary friction, form losses, pumps, turbines, hydraulic jumps, and energy dissipators. The total energy at a point on a streamline in Figure 6.5 (a) or (b) is equal to:

$$H = z + y + \frac{p}{\gamma} + \frac{V^2}{2g} \tag{6-6}$$

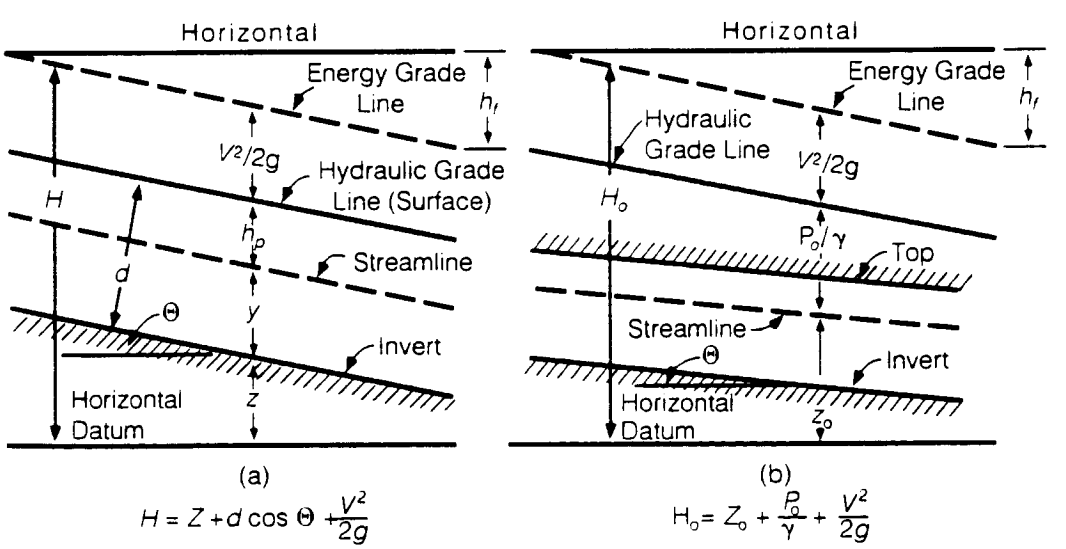

Figure 6.5—Definition of terms for total energy in an open channel and closed conduit: (a) open channel flow and (b) closed conduit flow (ASCE, 1982).

in which H is the total energy, z is the difference between the elevation of a point and some arbitrary horizontal datum (such as mean sea level), y is measured normal to the invert and is the difference in elevation between this point and the elevation of some streamline, p is the pressure on the streamline, γ is the fluid unit weight, v is the velocity of the streamline, and g is the acceleration due to gravity. The invert slope, θ, has been neglected in equation 6-6 ($\cos\theta$ is assumed to equal unity).

Since H is energy per unit weight, it has units of length and is often referred to as the total head or total energy in whatever length unit is used. The term p/γ is the pressure energy per unit weight and is known as the pressure head, h_p. Likewise, $V^2/2g$ is the kinetic energy per unit weight and is known as the velocity head, h_v.

Since z and y can be referenced to any streamline, it is sometimes convenient in closed conduit flow to let z be the difference in elevation between the centerline of the conduit and the elevation of the arbitrary datum and let y be zero. Therefore, for pressure flow, equation 6-6 becomes:

$$H = z + h_p + \frac{V^2}{2g} \tag{6-7}$$

where z is now the vertical distance to the conduit centerline.

For the open channel in Figure 6.5 (a) it is convenient to let z be the difference between the elevation of the channel invert and the elevation of the arbitrary datum, and to let d be the depth. The term p/γ is then zero. Therefore, for open channel flow, equation 6.6 becomes:

$$H = z + d\cos\theta + \left(\frac{V^2}{2g}\right) \tag{6-8}$$

for a plane section normal to the streamlines and with hydrostatic pressure distribution. The velocity V is the spatial average velocity in the section obtained as Q/A. As indicated above, the depth of flow is always measured normal to the channel bed and equals $d\cos\theta$ for the sketch in Figure 6.5 (a).

As in the earlier discussion for momentum, equation (6-8) applies to steady flow in storm sewers and man-made channels that have simple prismatic cross-sections and where any local velocity is approximately equal to the spatial average velocity in that section. In natural channels with floodplains and in complex man-made channels, the velocity head based on the average velocity V must be multiplied by a velocity distribution factor, α, to account for the variation in velocity across the cross-section. Equation 6-8 then becomes:

$$H = z + d\cos\theta + \alpha\left(\frac{V^2}{2g}\right) \tag{6-9}$$

Referring to Figure 6.5, the energy equation can now be written as:

$$z_1 + d_1 \cos\theta + \alpha \left(\frac{V_1^2}{2g}\right) = z_2 + d_2 \cos\theta + \alpha \left(\frac{V^2}{2g}\right) + H_L \qquad (6\text{-}10)$$

in which H_L is the sum of all the head losses between two sections 1 and 2 normal to the flow. These losses include major and minor losses. The major loss is that due to the friction of the water against the sides of the channel or pipe. The minor losses are called form losses and can include entrance, contraction, expansion, junction, exit, bend, and manhole losses. Major and minor losses will be discussed in more detail later.

A. Hydraulic and Energy Grade Lines

Two very useful concepts in flow analysis are the hydraulic and energy grade lines. These are shown in Figure 6.5 (a) and 6.5 (b) for open channel and pressure flow, respectively. For the case of pressure flow, the hydraulic grade line (HGL) is the piezometric surface, i.e. the height to which water will rise in a piezometer. It is often referred to as the piezometric head line (PHL). The energy grade line (EGL) is the line showing the total energy of the flow above some arbitrary horizontal datum. The slope of the EGL is called the energy slope or the friction slope and is designated S_f. The vertical difference between the HGL and the EGL is the velocity head.

For open channel flow, the HGL is equal to the water surface. As shown in Figure 6.5 (a), the sum of y and p/γ, the potential energy above the channel invert, is equal to d $\cos\theta$. If the slope of the channel invert is small, so that $\cos\theta$ can be assumed to be 1.00, then the potential energy above the invert can be assumed to be equal to the vertical depth of flow. This assumption is made for the remainder of this chapter, unless noted otherwise. Further, the velocity distribution coefficient is assumed equal to 1.00, unless otherwise noted.

B. Specific Energy

When the arbitrary horizontal datum is taken as the channel or pipe invert for open channel flow, Equation 6-8 becomes:

$$E = d + \left(\frac{V^2}{2g}\right) \qquad (6\text{-}11)$$

in which E is known as the specific energy per unit weight of fluid, usually expressed in feet. The specific energy curve giving E as a function of depth, d, has many uses and is constructed as shown in Example 6-1.

Example 6-1: Determine the specific energy curve for a rectangular channel 10 feet wide conveying a flow of 250 cfs. The calculations are shown in Table 6.1. Columns 1 and 5 are plotted to form a curve similar to that in Figure 6.6.

The depth corresponding to minimum specific energy is defined as critical depth, d_c. The region of flow above critical depth is known as the subcritical flow or tranquil zone. The region below critical depth is known as the supercritical or shooting flow zone. A vertical line through any specific energy greater than minimum specific energy will cut the curve at two locations, at a depth less than critical depth and at a depth greater than critical depth. These two depths are known as alternate depths.

One other aspect of the specific energy is worth noting. The nose of the curve near critical depth can be either somewhat pointed or flat depending on the cross-sectional shape. If it tends to be flat for a given situation, then the water surface will be unstable when depths are within 10–15% of critical depth because a slight change in energy may impart large fluctuations in either of the alternate depths.

C. Froude Number

When flow is at critical depth, specific energy equals E_{min}, the depth equals d_c, and the Froude number, F, is equal to one. If F is less than

TABLE 6.1. Example Calculation of Specific Energy in a Rectangular channel, width = 10 feet, Q = 250 cfs and α = 1.

Depth (ft.) (1)	Area (sq.ft.) (2)	Velocity (fps) (3)	$V^2/2g$ (ft.) (4)	E (ft.) (5)
0.0	0.0	—	—	—
0.5	5.0	50.00	38.87	39.37
1.0	10.0	25.00	9.72	10.72
1.5	15.0	16.67	4.32	5.82
2.0	20.0	12.50	2.43	4.43
2.5	25.0	10.00	1.55	4.05
2.69 (d_c)	-----	------	-------(E_{min})	4.03
3.0	30.0	8.33	1.08	4.08
3.5	35.0	7.14	0.79	4.29
4.0	40.0	6.25	0.61	4.61
4.5	45.0	5.56	0.48	4.98
5.0	50.0	5.00	0.39	5.39
5.5	55.0	4.54	0.32	5.82
6.0	60.0	4.17	0.27	6.27
6.5	65.0	3.85	0.23	6.73
7.0	70.0	3.57	0.20	7.20
7.5	75.0	3.33	0.17	7.67
8.0	80.0	3.12	0.15	8.15

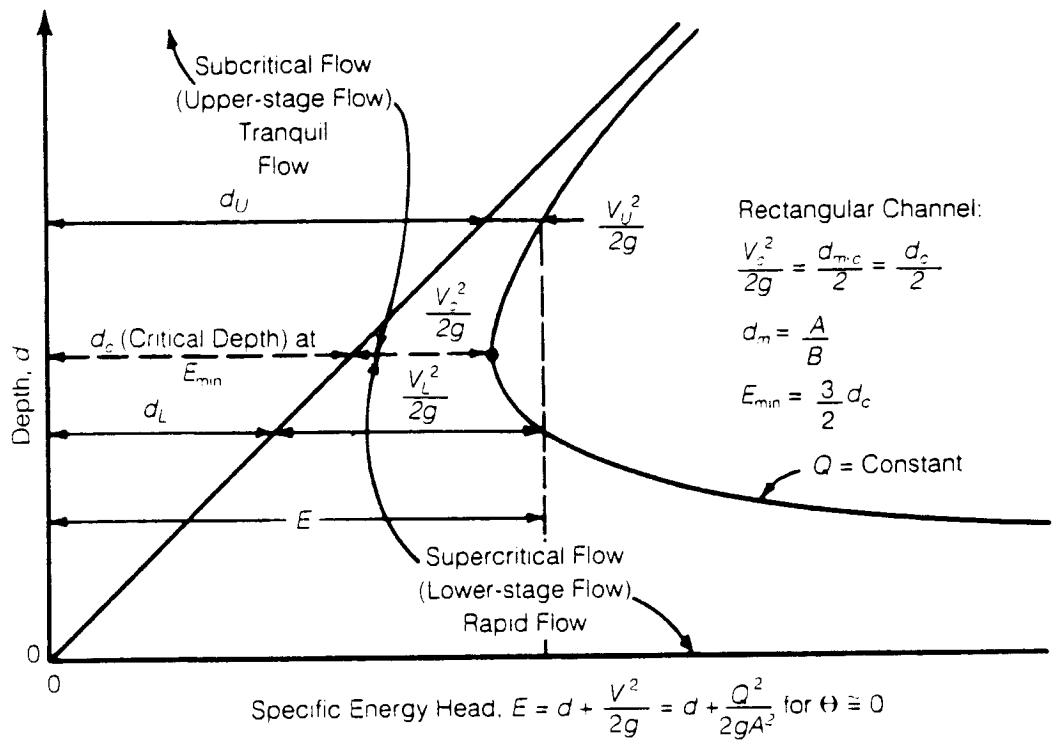

Figure 6.6—Specific energy curve for rectangular channel (ASCE, 1982).

one, then the flow is subcritical. If F is greater than one, the flow is supercritical. The Froude number is defined as:

$$F = \frac{V}{(gd_m)^{1/2}} \tag{6-12}$$

in which V is the spatial average velocity in the cross-section, g is the acceleration due to gravity, and d_m is the hydraulic mean depth, determined from:

$$d_m = \frac{A}{T} \tag{6-13}$$

in which A is the cross-sectional area and T is the top width of flow.

The primary significance of the Froude number in urban drainage design is that it represents the ratio of the average flow velocity to the propagation velocity of a small gravity wave and thus determines the direction of water surface profile computations. A gravity wave will be propagated in both the upstream and downstream directions in sub-

critical flow since the wave velocity is greater than the flow velocity, but can only be propagated downstream in supercritical flow. Consequently, water surface profile computations always begin at a control section and proceed upstream when the control depth is greater than d_c and downstream when the control depth is less than d_c. The Froude number also is used to detect the occurrence of a hydraulic jump forming in a channel or conduit.

D. Critical Depth

As noted above, critical depth is defined as the depth at the point of minimum specific energy for constant discharge. Critical depth is a function of discharge, size of channel, and shape of channel. Its value can be calculated in one of three ways: (1) Equation 6-14 for an irregular channel of any cross-section; (2) from tables contained in French (1985) for rectangular and trapezoidal channels; and (3) from nomographs contained in FHWA (1981). Equation 6-14 is a general expression for critical depth in a channel of any cross-sectional shape:

$$\alpha \left(\frac{Q^2}{g}\right) = \frac{A^3}{T} \tag{6-14}$$

in which Q is the discharge, g is the acceleration due to gravity, A is the cross-sectional area of flow, and T is the top width of flow.

Example 6-2: Use the above three methods to calculate critical depth for the rectangular channel described in Example 6-1, with Q = 250 cfs, b = 10 feet, and a = 1.

Method 1: Equation (6-14)

$$\frac{Q^2}{g} = \frac{A^3}{T} = \frac{b^3 d_c^3}{b} = b^2 d_c^3$$

$$(250)^2/32.15 = (10)^2\,(d_c)^3$$

$$d_c = (62{,}500/3215)^{1/3}$$

$$= 2.69 \text{ feet}$$

Alternatively, for a rectangular channel, Equation 6-14 reduces to:

$$d_c = \left(\frac{q^2}{g}\right)^{1/3} \tag{6-15}$$

where q = discharge per unit width in cfs/ft. Solving (6-15) yields d_c = 2.69 feet.

Method 2: Table of K_c' values for trapezoidal channels.

$$K'_c = \frac{Q}{b^{5/2}}$$

$$= 250/316 = 0.790$$

Table 6.2 is taken from Brater and King (1976), and for a rectangular channel, the value of $K_c' = 0.790$ lies between d_c/b equal to 0.26 and 0.27. Linear interpolation produces:

$$\frac{d_c}{b} = 0.269$$

$$d_c = 0.269 \times 10 = 2.69 \text{ feet}$$

Method 3: Nomograph (FHWA, 1961)
Figure 6.7 is a nomograph from FHWA (1961), which includes other nomographs for circular and trapezoidal channels. From Figure 6.7, for a discharge of 250 cfs, critical depth is equal to 2.7 feet. Figure 6.7 also can be used to determine the normal depth as noted in the following section.

Figure 6.8 is a nomograph for critical depth and velocity in circular conduits. The nomograph solutions in Figures 6.7 and 6.8 are presented to illustrate the ease of graphical solution to problems of computing critical and normal depth. Programmable calculators and microcomputers can also be used to obtain iterative numerical solutions. Refer to Croley (1979) and Smith (1986) for details of computational programs pertinent to storm drainage hydraulics.

VI. NORMAL DEPTH

Normal depth is defined simply as the depth of uniform flow under constant discharge. Recall that in uniform flow the losses due to boundary friction are just balanced by the gravity component in the direction of flow. In other words, friction and gravity forces in the direction of flow are equal but act in opposite directions. At normal depth the slope of the invert, the slope of the HGL, and the slope of the EGL are numerically equal and parallel to each other.

Normal depth is a function of discharge, size of channel, shape of channel, slope of channel, and frictional resistance to flow. Its value can be calculated in one of three ways: (1) by the Manning equation (6-16) and Equation (6-17); (2) by tables contained in French (1985); and (3) by nomographs contained in FHWA (1961). Equation (6-16) is Manning's Equation:

$$V = \frac{1.49}{n} R^{2/3} S_f^{1/2} \qquad \text{(6-16a)}$$

TABLE 6.2. Values of K_c' in the Formula $Q = K_c' b^{5/2}$ for Critical Depth in Trapezoidal Channels (Brater and King 1976).

$\frac{D_c}{b}$	Side slopes of channel, ratio of horizontal to vertical									
	Vertical	¼–1	½–1	¾–1	1–1	1½–1	2–1	2½–1	3–1	4–1
.01	.0057	.0057	.0057	.0057	.0057	.0057	.0057	.0057	.0058	.0058
.02	.0160	.0161	.0161	.0162	.0162	.0163	.0164	.0165	.0165	.0167
.03	.0295	.0296	.0297	.0298	.0299	.0302	.0304	.0306	.0309	.0314
.04	.0454	.0456	.0458	.0461	.0463	.0468	.0473	.0478	.0483	.0493
.05	.0634	.0638	.0642	.0646	.0650	.0659	.0668	.0677	.0686	.0704
.06	.0833	.0840	.0846	.0853	.0859	.0873	.0887	.0902	.0916	.0946
.07	.1050	.1060	.1069	.1079	.1089	.1109	.1130	.1151	.1173	.1218
.08	.1283	.1296	.1310	.1323	.1337	.1366	.1395	.1426	.1456	.1520
.09	.1531	.1549	.1567	.1585	.1604	.1643	.1683	.1724	.1766	.1852
.10	.1793	.1816	.1840	.1864	.1889	.1940	.1992	.2046	.2101	.2214
.11	.2069	.2098	.2128	.2159	.2191	.2256	.2323	.2392	.2463	.2607
.12	.2357	.2394	.2431	.2470	.2509	.2591	.2676	.2762	.2851	.3032
.13	.2658	.2702	.2748	.2796	.2844	.2945	.3049	.3156	.3265	.3488
.14	.2971	.3024	.3079	.3137	.3196	.3318	.3444	.3574	.3706	.3975
.15	.3295	.3358	.3424	.3493	.3563	.3710	.3861	.4015	.4173	.4495
.16	.363	.370	.378	.386	.395	.412	.430	.448	.467	.505
.17	.397	.406	.415	.425	.435	.455	.476	.497	.519	.563
.18	.433	.443	.454	.465	.476	.499	.524	.549	.547	.625
.19	.470	.481	.493	.506	.519	.546	.574	.603	.632	.691
.20	.507	.520	.534	.549	.564	.594	.626	.659	.692	.760
.21	.546	.561	.576	.593	.610	.645	.681	.718	.755	.832
.22	.585	.602	.620	.638	.657	.697	.737	.779	.822	.908
.23	.626	.644	.664	.685	.706	.751	.796	.843	.891	.988
.24	.667	.688	.710	.733	.757	.806	.858	.910	.963	1.071
.25	.709	.732	.757	.782	.809	.864	.921	.979	1.038	1.158
.26	.752	.777	.805	.833	.862	.923	.986	1.051	1.116	1.248
.27	.796	.824	.854	.885	.918	.985	1.054	1.125	1.197	1.343
.28	.840	.871	.904	.938	.974	1.048	1.124	1.202	1.281	1.441
.29	.886	.919	.955	.993	1.032	1.113	1.197	1.283	1.368	1.543
.30	.932	.969	1.008	1.049	1.092	1.180	1.272	1.365	1.458	1.649
.31	.979	1.019	1.062	1.107	1.153	1.249	1.349	1.450	1.552	1.759
.32	1.027	1.070	1.116	1.165	1.216	1.320	1.428	1.537	1.648	1.873
.33	1.075	1.122	1.172	1.225	1.280	1.393	1.510	1.628	1.748	1.991
.34	1.124	1.175	1.229	1.286	1.345	1.468	1.594	1.722	1.851	2.113
.35	1.174	1.229	1.287	1.349	1.413	1.545	1.680	1.818	1.958	2.240
.36	1.225	1.283	1.346	1.413	1.481	1.623	1.769	1.917	2.067	2.370
.37	1.276	1.339	1.407	1.478	1.552	1.704	1.860	2.019	2.180	2.505
.38	1.328	1.395	1.468	1.544	1.623	1.786	1.954	2.124	2.296	2.644
.39	1.381	1.453	1.530	1.612	1.697	1.871	2.050	2.232	2.416	2.787
.40	1.435	1.511	1.594	1.681	1.771	1.958	2.149	2.343	2.539	2.935
.41	1.489	1.570	1.658	1.752	1.848	2.046	2.250	2.457	2.665	3.087
.42	1.544	1.630	1.724	1.823	1.926	2.137	2.353	2.573	2.795	3.243
.43	1.599	1.691	1.791	1.896	2.005	2.229	2.460	2.693	2.929	3.404
.44	1.655	1.752	1.859	1.970	2.086	2.324	2.568	2.816	3.066	3.570
.45	1.712	1.815	1.928	2.046	2.168	2.421	2.679	2.942	3.206	3.740

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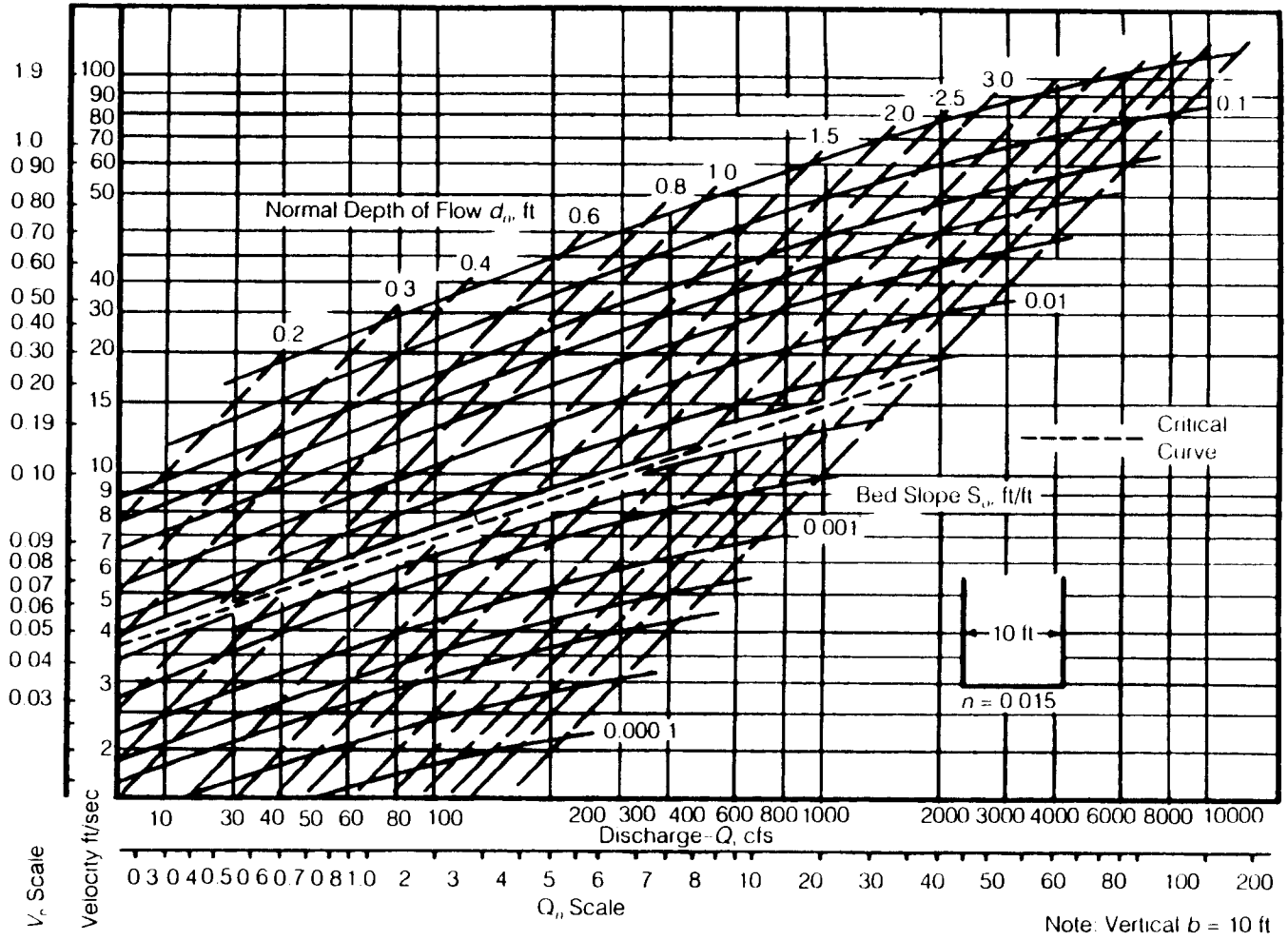

Figure 6.7—Normal and critical depth for uniform flow in rectangular channel (cfs × 0.028 32 = m³/s; ft × 0.304 8 = m; and ft/sec × 0.304 8 = m/s) (FHWA, 1961).

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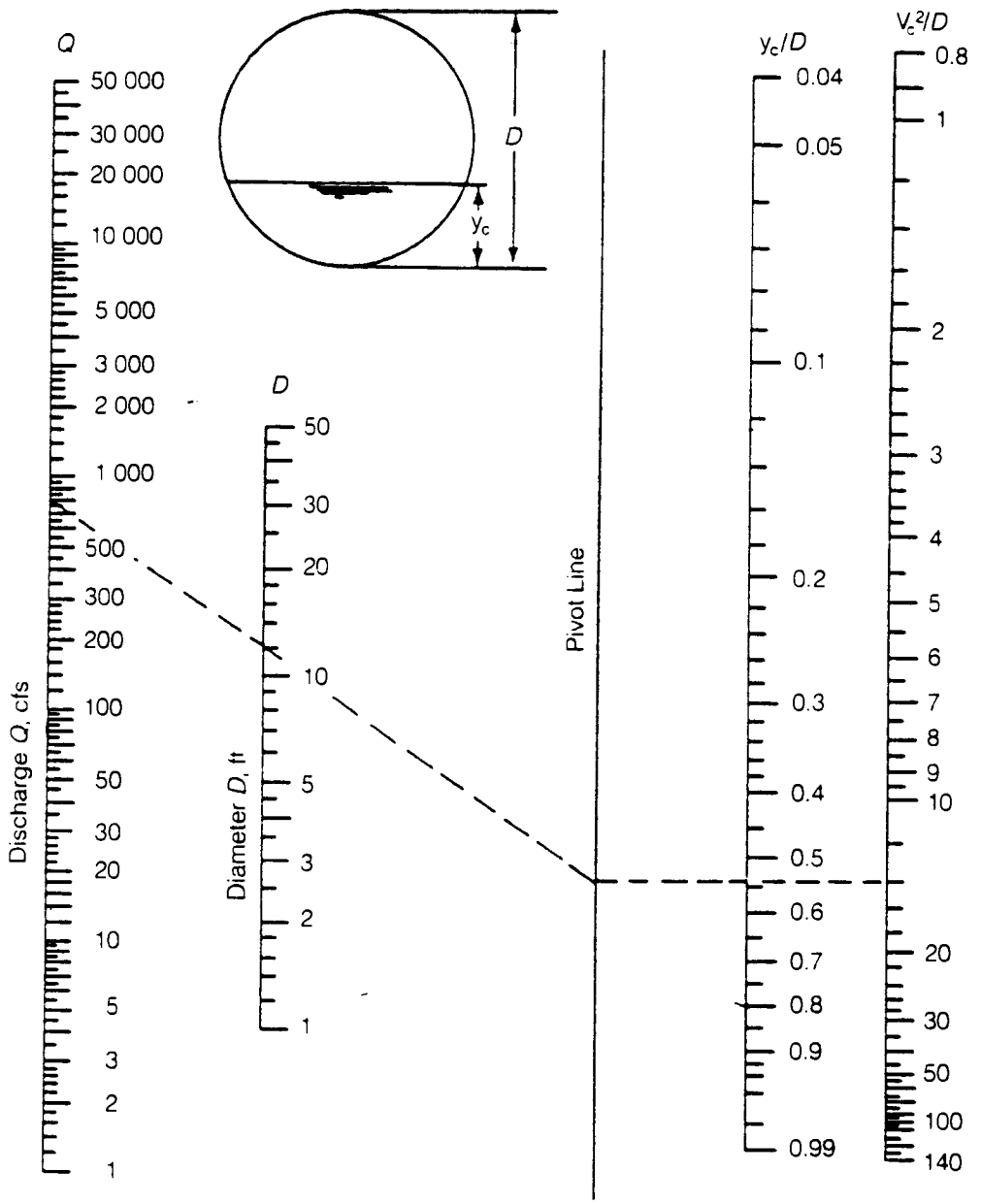

Figure 6.8—Critical flow and critical velocity in circular conduits. Given Q = *800 cfs and* D = *12 ft; enter* Q *scale at* Q = *800 cfs, follow broken line, and turn at the pivot line:* y_c/D = *0.54,* y_c = *6.48 ft,* V_c^2/D = *14, and* V_c = *12.95 fps (cfs × 0.028 32 =* m^3/s*, ft × 0.304 8 = m, and fps × 0.304 8 = m/s) (AISI, 1985).*

in which V is the average velocity of flow in feet per second, R is the hydraulic radius in feet, S_f is the friction slope in feet per foot, and n is Manning's roughness coefficient. The hydraulic radius, R, in Manning's Equation is A/WP, where A is the cross-sectional area of flow in square feet and WP is the wetted perimeter in feet (the perimeter of the channel wetted by water, excluding the free surface width).

In the SI system of units, Manning's Equation is:

$$V = \frac{1}{n} R^{2/3} S_f^{1/2} \tag{6-16b}$$

where V is in meters per second and R is in meters. It should be noted that Mannings n in SI units = $0.6730n$ in English units of measure.

As noted, S_f is the friction slope, or the slope of the EGL. When S_f is set equal to the slope of the channel, S_o, the resulting depth of flow calculated from Manning's equation is the normal depth. Since $Q = AV$, Manning's Equation can be written as:

$$Q = \frac{1.49}{n} AR^{2/3} S_o^{1/2} \tag{6-17}$$

Equation 6-17 sometimes is written as:

$$q = k(S_f^{1/2}) \tag{6-18}$$

where

$$K = \frac{1.49}{n} AR^{2/3} \tag{6-19}$$

and is called the conveyance of the channel section.

Example 6-3: Use the three methods shown in the previous section to calculate normal depth for the flow situation described in Example 6-1 where Q = 250 cfs in a 10-foot wide rectangular channel. Assume the slope of the channel is 0.2 percent and Manning's n is 0.014.

Method 1
Manning's equation is:

$$Q = \frac{1.49}{n} AR^{2/3} S_o^{1/2}$$

$AR^{2/3}$ is a function of the geometric properties of the channel. Solving for $AR^{2/3}$:

$$A^{2/3} = \frac{Qn}{1.49 S_o^{1/2}}$$

$$= 250 \times \frac{0.014}{1.49(0.002)^{1/2}}$$

$$= 52.53$$

TABLE 6.3. Calculations for Normal Depth Using Manning's Equation.

Depth (ft.) (1)	Area (sq. ft.) (2)	WP (feet) (3)	R (ft.) (4)	$R^{2/3}$ (5)	$AR^{2/3}$ (6)
5.00	50.0	20.00	2.50	1.84	92.1
4.00	40.0	18.00	2.22	1.70	68.1
3.50	35.0	17.00	2.06	1.62	56.6
3.40	34.0	16.80	2.02	1.50	54.4
3.31	33.1	16.62	1.99	1.58	52.4
3.32	33.2	16.64	2.00	1.58	52.6

Normal depth = 3.32 feet = d_n.

Using the relationships $A = bd$, $WP = b + 2d$, and $R = A/WP$, determine the value of normal depth by trial and error as shown in Table 6.3.

Method 2

Table 6.4 is a portion of Table 7.11 from Brater and King (1976).

$$K' = \frac{Qn}{b^{8/3}S^{1/2}}$$

$$= 250 \times \frac{0.014}{10^{8/3}(0.002)^{1/2}}$$

$$= 250 \times \frac{0.014}{464} \times 0.0447$$

$$= 0.1686$$

From Table 6.4, for a rectangular channel, the value of $K' = 0.1686$ lies between d_n/b equal to 0.33 and 0.34. Linear interpolation gives:

$$d_n/b = 0.332$$

$$d_n = 0.332 \times 10 = 3.32 \text{ feet}$$

Method 3

Figure 6.7 is a nomograph from FHWA (1961). Other nomographs for d_n are presented there for circular and trapezoidal channels. Note that Figure 6.7 is set for a value of n equal to 0.012. Therefore, in this case, we must use the Qn scale. For a discharge of 250 cfs and n equal to 0.014, Qn is equal to 3.5. Using this value and a channel slope of 0.002, normal depth is equal to 3.3 feet.

TABLE 6.4. Values of K' in formula $Q = \frac{k'}{n} b^{2/3} s^{1/2}$ for trapezoidal channels (Brater and King 1976)

$\frac{D}{b}$	Side slopes of channel, ratio of horizontal to vertical									
	Vertical	¼–1	½–1	¾–1	1–1	1½–1	2–1	2½–1	3–1	4–1
.01	.00068	.00068	.00069	.00069	.00069	.00069	.00069	.00069	.00070	.00070
.02	.00213	.00215	.00216	.00217	.00218	.00220	.00221	.00222	.00223	.00225
.03	.00414	.00419	.00423	.00426	.00428	.00433	.00436	.00439	.00443	.00449
.04	.00660	.00670	.00679	.00685	.00691	.00700	.00708	.00716	.00723	.00736
.05	.00946	.00964	.00979	.00991	.01002	.01019	.01033	.01047	.01060	.01086
.06	.0127	.0130	.0132	.0134	.0136	.0138	.0141	.0143	.0145	.0150
.07	.0162	.0166	.0170	.0173	.0175	.0180	.0183	.0187	.0190	.0197
.08	.0200	.0206	.0211	.0215	.0219	.0225	.0231	.0236	.0240	.0250
.09	.0241	.0249	.0256	.0262	.0267	.0275	.0282	.0289	.0296	.0310
.10	.0284	.0294	.0304	.0311	.0318	.0329	.0339	.0348	.0358	.0376
.11	.0329	.0343	.0354	.0364	.0373	.0387	.0400	.0413	.0424	.0448
.12	.0376	.0393	.0408	.0420	.0431	.0450	.0466	.0482	.0497	.0527
.13	.0425	.0446	.0464	.0480	.0493	.0516	.0537	.0556	.0575	.0613
.14	.0476	.0502	.0524	.0542	.0559	.0587	.0612	.0636	.0659	.0706
.15	.0528	.0559	.0585	.0608	.0627	.0662	.0692	.0721	.0749	.0805
.16	.0582	.0619	.0650	.0676	.0700	.0740	.0777	.0811	.0845	.0912
.17	.0638	.0680	.0716	.0748	.0775	.0823	.0866	.0907	.0947	.1026
.18	.0695	.0744	.0786	.0822	.0854	.0910	.0960	.1008	.1055	.1148
.19	.0753	.0809	.0857	.0899	.0936	.1001	.1059	.1115	.1169	.1277
.20	.0812	.0876	.0931	.0979	.1021	.1096	.1163	.1227	.1290	.1414
.21	.0873	.0945	.101	.106	.111	.120	.127	.135	.142	.156
.22	.0934	.1015	.109	.115	.120	.130	.139	.147	.155	.171
.23	.0997	.1087	.117	.124	.130	.141	.150	.160	.169	.187
.24	.1061	.1161	.125	.133	.140	.152	.163	.173	.184	.204
.25	.1125	.1236	.133	.142	.150	.163	.176	.188	.199	.222
.26	.119	.131	.142	.152	.160	.175	.189	.202	.215	.241
.27	.126	.139	.151	.162	.171	.188	.203	.218	.232	.260
.28	.132	.147	.160	.172	.182	.201	.217	.234	.249	.281
.29	.139	.155	.170	.182	.194	.214	.232	.250	.268	.302
.30	.146	.163	.179	.193	.205	.228	.248	.267	.287	.324
.31	.153	.172	.189	.204	.218	.242	.264	.285	.306	.347
.32	.160	.180	.199	.215	.230	.256	.281	.304	.327	.371
.33	.167	.189	.209	.227	.243	.271	.298	.323	.348	.396
.34	.174	.198	.219	.238	.256	.287	.316	.343	.370	.423
.35	.181	.207	.230	.251	.269	.303	.334	.363	.392	.450
.36	.189	.216	.241	.263	.283	.319	.353	.385	.416	.478
.37	.196	.225	.252	.275	.297	.336	.372	.406	.440	.507
.38	.203	.234	.263	.288	.312	.353	.392	.429	.465	.537
.39	.211	.244	.274	.301	.326	.371	.413	.452	.491	.568
.40	.218	.253	.286	.315	.341	.389	.434	.476	.518	.600
.41	.226	.263	.297	.328	.357	.408	.456	.501	.546	.633
.42	.233	.273	.309	.342	.373	.427	.478	.526	.574	.668
.43	.241	.283	.321	.357	.389	.447	.501	.553	.603	.703
.44	.248	.293	.334	.371	.405	.467	.525	.580	.633	.740
.45	.256	.303	.346	.386	.422	.488	.549	.607	.664	.777

D = depth of water b = bottom width of channel

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VII. WATER SURFACE PROFILES

As noted in the first section of this chapter, open channel flow can be classified as uniform or nonuniform. Uniform flow is characterized by normal depth computed from Manning's equation. Nonuniform flow, on the other hand, will be either gradually or rapidly varied. Gradually varied flow can be analyzed by making the following assumptions:

(a) The slope of the EGL at a cross-section in a channel is the same as it would be for uniform flow at that section with the same velocity and hydraulic radius.
(b) The vertical curvature of the streamlines is small enough for the pressure in the flow to be hydrostatic in plane sections normal to the bed.
(c) The channel is prismatic (of constant shape with straight alignment or negligible curvature and constant bed slope).
(d) The momentum and energy correction factors are constant along the length of channel or pipe.

Rapidly varied flow by contrast does not conform to the above conditions. It occurs in abrupt transitions such as flow over weirs, under gates, and through junctions. The hydraulic jump is perhaps the best example of rapidly varied flow of interest in urban drainage design. Rapidly varied flow has the following characteristics:

(a) The local curvature of streamlines in the flow is so pronounced that the pressure distribution of the flow is not hydrostatic.
(b) A rapid variation in flow velocity and depth occurs within a relatively short reach of the channel.
(c) Separation zones, eddies, and rollers may occur and complicate flow patterns or distort the velocity distribution.
(d) The energy and momentum correction factors vary greatly and may be difficult to determine.

The identification of particular water surface profiles in steady gradually varied flow is determined by the relative magnitudes of the actual flow depth, the normal depth, the critical depth, and the location of control sections. Water surface profiles in gradually varied flow are characterized first by one of the following types that describe the slope of the channel.

M—Mild	:	$d_n > d_c$
C—Critical	:	$d_n = d_c$
S—Steep	:	$d_n < d_c$
H—Horizontal	:	d_n does not exist
A—Adverse	:	d_n does not exist

Water surface profiles are then characterized by the location of the actual depth of flow in relation to normal and critical depth, according to the following zones:

Zone 1—depth of flow is greater than d_n and d_c
Zone 2—depth of flow is between d_n and d_c
Zone 3—depth of flow is less than d_n and d_c

Thus, a water surface profile might be classified as M-1 (mild slope, zone 1), S-2 (steep slope, zone 2) etc. **Preliminary identification of the water surface profile type is an important first step in actual computation of water surface elevations in a channel or major storm sewer.**

A water surface profile is computed by the following procedure.

(a) Assemble the basic hydraulic data for each channel segment, including slope, width, side slopes, roughness, and discharge.
(b) Establish the location of "control" sections where flow is a unique function of depth, such as at a natural falls or the crest of a spillway.
(c) Determine d_n and d_c for each channel segment and plot on rough profile.
(d) Locate all the control sections on the profile and sketch the possible water surface profiles. Recall that transition from supercritical to subcritical flow is only possible in a hydraulic jump.
(e) Compute the actual water surface elevations by applying the energy equation to individual sections of the channel above or below the control.

The actual calculation of water surface profile is made by direct step or standard step methods, as discussed later in this chapter. Both methods proceed from a control section. A control is any section in a channel where the depth of flow is known, such as critical depth, depth upstream of a culvert, depth of flow over a weir or dam, or depth of flow under a gate. If the flow above the control is subcritical, then the water surface profile calculation will proceed in the upstream direction from the control. If flow below the control is supercritical, then the calculations proceed downstream. This latter situation is illustrated in Figure 6.9(a), while the former occurs in Figure 6.9(b). Most water surface profiles take the form of a draw-down, Figure 6.9(a), or a backwater curve, Figure 6.9(b).

Figure 6.10 depicts three water surface profiles for gradually varied flow on mild and steep slopes. Other profiles are discussed in the following paragraphs. A more complete listing of water surface profile types with detailed discussion can be found in basic hydraulic texts (Chow 1959, Brater and King 1976, Rouse 1961). One of the most common water surface profiles is the M-1 profile shown in Figure 6.11. Here water in a channel on a mild slope flows into a pool. The pool could be the water surface upstream of a dam, a culvert, a bridge, or at the upstream end of a detention basin. If the channel is on a hydraulically steep slope instead of a mild slope, the water surface profile will show a hydraulic jump. A hydraulic jump occurs whenever flow passes through critical depth enroute from a hydraulically steep to a hydraulically mild slope.

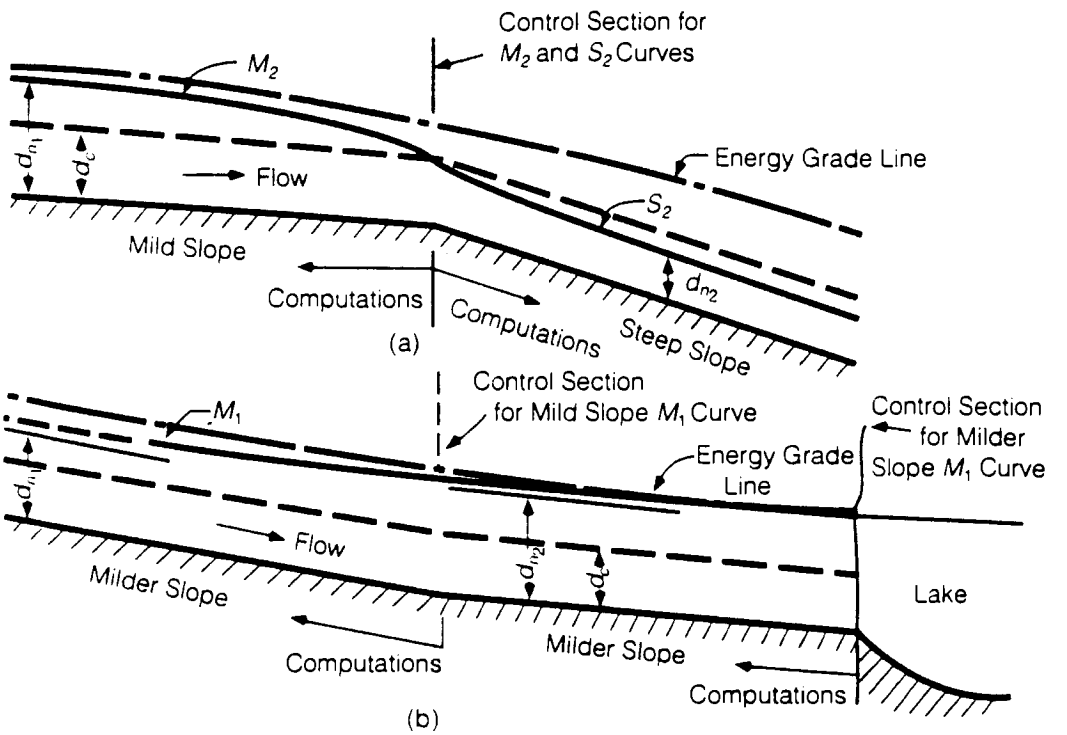
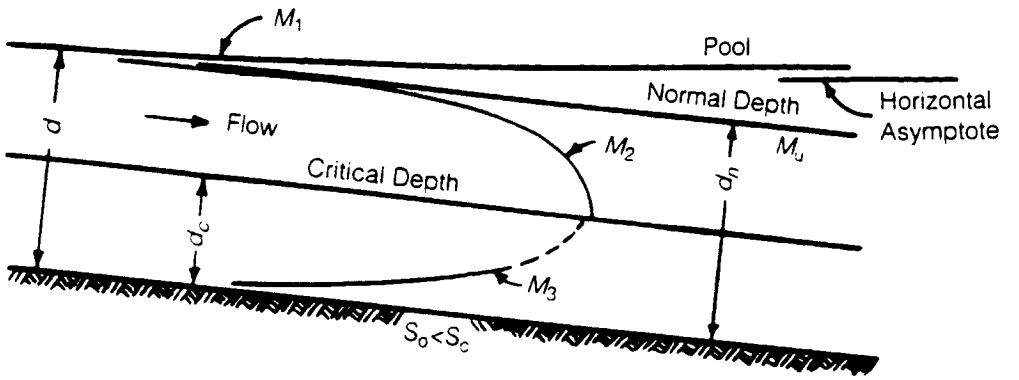


Figure 6.9—Drawdown and backwater profiles in gradually varied flow: (a) drawdown mild to steep slope and (b) backwater mild to milder slope (ASCE, 1982).

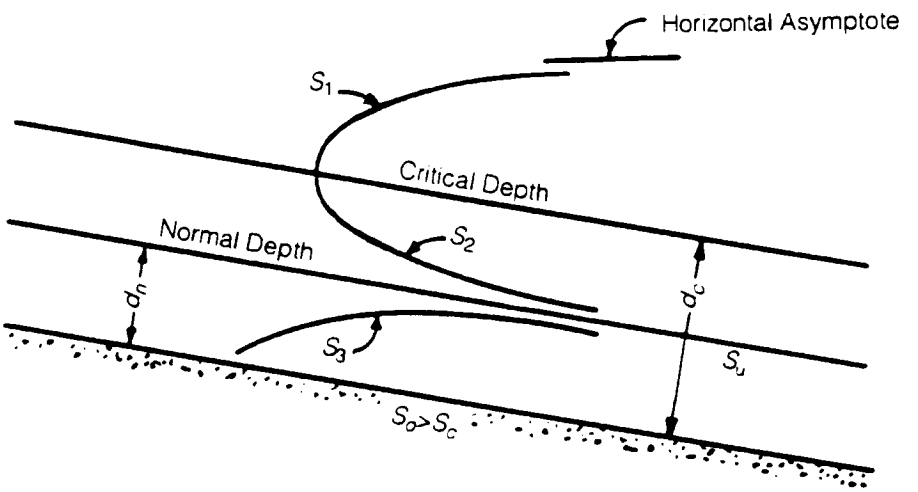
VIII. HYDRAULIC JUMP

The hydraulic jump is a rapidly varied flow phenomenon in which flow in a channel changes abruptly from supercritical flow at a relatively shallow depth (less than d_c) to subcritical flow at a greater depth (greater than d_c). The depth before the jump is called the initial depth, while the depth after the jump is known as the sequent depth. The situation is illustrated in Figure 6.12.

The hydraulic jump may be employed as a device for the dissipation of excess energy, as where a steep sewer enters a larger sewer at a junction. In stormwater projects, the hydraulic jump may be used to consume excess energy and avoid scour of earthen channels. Thus, the analysis of hydraulic jumps usually has three objectives. First, the location of the jump is important because of the potential for unexpected surcharge or channel scour. This can be determined by searching for pipe/channel elements where the flow is supercritical upstream and subcritical downstream. Once this is determined, it is important to compute the two depths, d_1 and d_2, which are the initial and sequent depths, respectively. Third, the energy loss H_j dissipated by the jump



M_u—Uniform Flow
M_1—Backwater from Resevoir or from Channel of Milder Slope ($d>d_n$);
M_2—Drawdown, as from Change of Channel of Mild Slope to Steep Slope ($d_n>d>d_c$); and
M_3—Flow Under Gate on Mild Slope, or Upstream Profile Before Hydraulic Jump on Mild Slope ($d<d_c$).



S_u—Uniform Flow
S_1—Downstream Profile After Hydraulic Jump on Steep Slope ($d>d_c$);
S_2—Drawdown, as from Mild to Steep Slope or Steep Slope to Steeper Slope ($d_c>d>d_n$); and
S_3—Flow Under Gate on Steep Slope, or Change from Steep Slope to Less Steep Slope ($d<d_n$).

Figure 6.10—Water surface profiles for gradually varied flow on (a) mild and (b) steep slopes (ASCE, 1982).

is often an important design consideration. The pertinent depth equation for a rectangular section is:

$$\frac{d_2}{d_1} = 0.5[(1 + 8F_1^2)^{1/2} - 1] \tag{6-20}$$

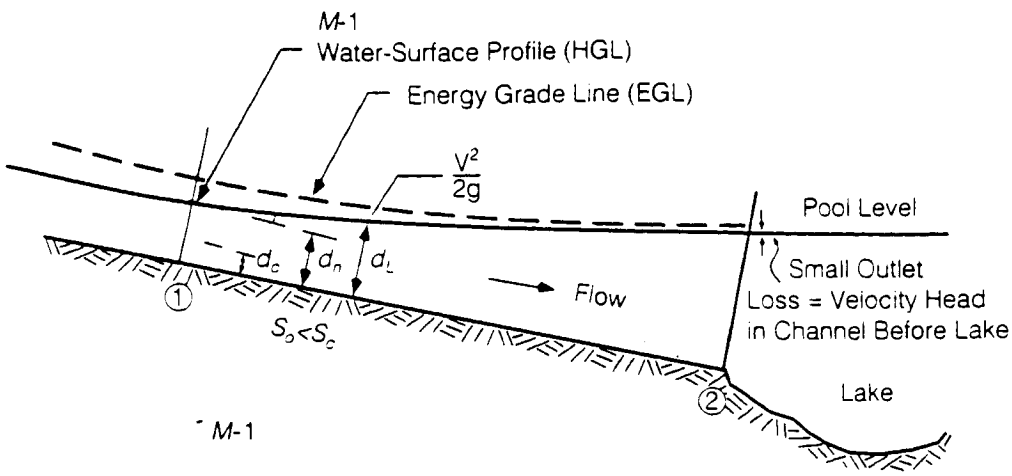


Figure 6.11—Water-surface profile in flow from a channel to a pool on a mild slope. Note that the water surface should approach d_n *asymptotically (FHWA, 1961).*

in which F_1 is the Froude number at the upstream section. The energy lost in the jump, H_j, is obtained by subtracting the specific energy at section 2 in Figure 6.12 from that at section 1.

$$H_j = E_1 - E_2 = \frac{(d_2 - d_1)^3}{(4d_1d_2)} \tag{6-21}$$

Equations similar to (6-20) and (6-21) are readily derived from energy conservation principles for other section geometries.

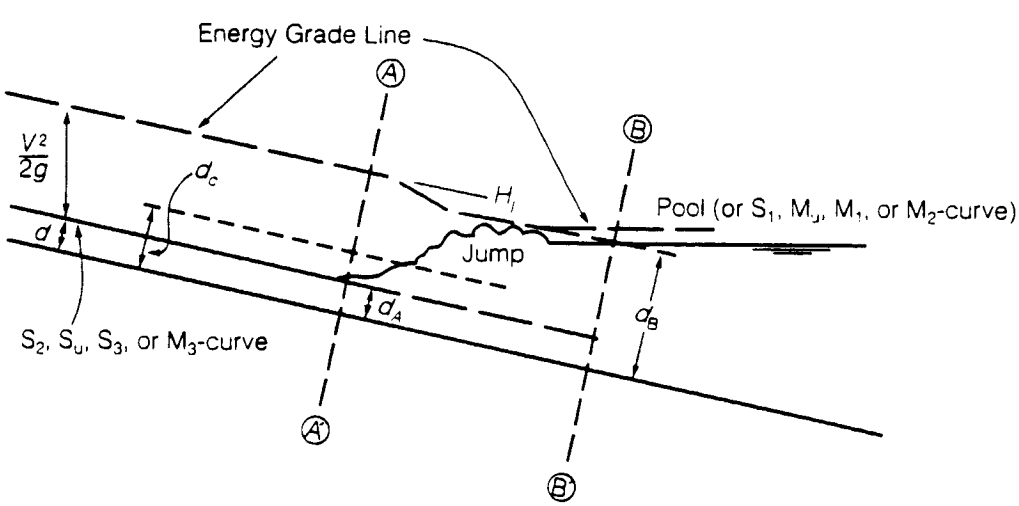


Figure 6.12—Water-surface profile for hydraulic jump: A–A′ = initial section and B–B′ = sequent section (FHWA, 1961).

IX. FRICTION LOSSES

The major loss in a channel or pipe is the friction or boundary shear loss. The head loss due to friction in a pipe or channel is computed from the general definition:

$$H_f = L x \overline{S}_f \tag{6-22}$$

in which H_f is the head loss due to friction, L is the length of channel or conduit, $\overline{S}_f$ is the average friction slope for the length L, and the subscripts 1 and 2 refer to the ends of the reach. The average friction slope $\overline{S}_f$ can be computed as the simple mean $(S_{f1} + S_{f2})/2$, or it can be evaluated from the mean depth or conveyance for the entire reach length. The friction slope between sections 1 and 2 at a single point along a pipe or channel is computed from one of several so-called friction formulas developed for uniform and gradually varied flow. The present practice is to use the Hazen-Williams formula for closed conduit or pressure flow when flow is in the hydraulically smooth range, and to use the Manning's equation in open channel and pipe flow when flow is in the transition and hydraulically rough range. The Hazen-Williams formula may be used when:

$$\frac{V_f(\kappa)}{\upsilon} < 3 \tag{6-23}$$

and

$$\frac{V_m(R)}{\upsilon} > 30{,}000 \tag{6-24}$$

where V_f equals the shear velocity $(g\ R\ S_f)^{0.5}$, κ is equivalent sand roughness, υ is the kinematic viscosity, V_m is the mean velocity, R is the hydraulic radius. For flow with Reynolds numbers less than 30,000, other formulas such as the Blasius formula may be used in the hydraulically smooth range. Christensen (1985) suggests the Manning Equation is valid when:

$$\frac{V_f(\kappa)}{\upsilon} > 3 \tag{6-25}$$

and

$$5 < \frac{R}{K} < 300 \tag{6-26}$$

According to French (1985) fully rough flow and Manning's Equation apply when:

$$n^6\sqrt{RS_f} \geq 1.9 \times 10^{-13} \tag{6-27}$$

where n is the Manning roughness coefficient. As a matter of interest consider a 24-inch concrete sewer flowing full at a 1% slope with n = 0.012. The pipe carries a discharge of 25 cfs. If we say that $S_f = S_o$ and that κ, the equivalent sand roughness, equals 0.01 feet for precast concrete pipe the following results are obtained with = 1.217 x 10^{-5} ft^2/sec for water at 60°F:

$$\frac{V_f(\kappa)}{\upsilon} = \frac{0.40 \times 0.01}{1.217 \times 10^{-5}} = 329$$

$$\frac{V_m(R)}{\upsilon} = \frac{7.96 \times 0.5}{1.217 \times 10^{-5}} = 326{,}941$$

$$\frac{R}{K} = \frac{0.50}{0.01} = 50$$

$$n^6\sqrt{RS_f} = (0.012)^6(0.5 \times 0.01)^{0.5} = 50$$

Comparing these parameter values with Equations 6-23 through 6-27 it seems we are clearly in the fully rough zone where the Manning Equation should be valid. As a practical matter, we would continue the analysis of friction head loss in this particular pipe by the Manning equation. A brief discussion of the two principal friction loss formulas follows.

A. Hazen-Williams Formula

The Hazen-Williams formula for smooth flow in a pipe is:

$$V = 1.318C_{HW}R^{0.63}S_f^{0.54} \tag{6-28a}$$

where V is the mean flow velocity in feet per second, C_{HW} is the Hazen-Williams coefficient, and all other terms are as defined previously.

The Hazen-Williams formula in SI units is:

$$V = 0.85C_{HW}R^{0.63}S_f^{0.54} \tag{6-28b}$$

where V is in meters per second.

The following are values of C_{HW} suggested by Brater and King (1976) for pipes carrying water. The C_{HW} values for new, smooth pipes generally are taken to be from 130 to 140. To estimate friction losses for

future conditions, lower values of C_{HW} are used to allow for reductions in flow capacity resulting from the factors listed above. For smooth concrete and cement-lined pipes, a C_{HW} value of 100–120 commonly is used for future conditions. A wide variety of values published in table form are available (Williams and Hazen 1945). For the example cited above, the friction slope computed by the Hazen-Williams equation, assuming C_{HW} = 120, is:

$$V = \frac{Q}{A} = \frac{25}{3.1416} = 7.96 \text{ ft/sec}$$

From Equation (6-28):

$$S_f = \left(\frac{V}{1.318 C_{HW} R^{0.63}}\right)^{1.852}$$

$$S_f = \left(\frac{7.96}{1.318 \times 120 \times 0.5^{0.63}}\right)^{1.852} \qquad (6\text{-}29)$$

$$= 0.0088 \text{ ft/ft}$$

The head loss, H_f, in 100 feet of precast concrete storm sewer would be 0.88 feet.

B. Darcy-Weisbach Equation

The Darcy-Weisbach Equation, also developed primarily for flow in pipes, is:

$$H_f = f \frac{L}{d_o} \frac{v^2}{2g} \qquad (6\text{-}30)$$

where:

f = friction factor
L = length of pipe, feet
d_o = diameter of pipe, feet

C. Manning Equation

The Manning Equation 6-16 is used widely in analyzing uniform and gradually varied flow in pipes and open channels. Typical values of "n" for both closed conduits and open channels are listed in Table 6.5. More complete listings of n values are found in Chow (1959) and FHWA (1961). The indirect effect of depth of flow or the height of vegetal cover on the roughness coefficient has been investigated in grass-lined chan-

TABLE 6.5. Values of Manning Coefficient for Various Materials (ASCE 1982).

Conduit Material (1)	Manning n (2)[a]
Closed conduits	
Asbestos-cement pipe	0.011–0.015
Brick	0.013–0.017
Cast iron pipe	
Cement-lined & seal coated	0.011–0.015
Concrete (monolithic)	
Smooth forms	0.012–0.014
Rough forms	0.015–0.017
Concrete pipe	0.011–0.015
Corrugated-metal pipe	
(½-in. × 2½-in. corrugations)	
Plain	0.022–0.026
Paved invert	0.018–0.022
Spun asphalt lined	0.011–0.015
Plastic pipe (smooth)	0.011–0.015
Vitrified clay	
Pipes	0.011–0.015
Liner plates	0.013–0.017
Open channels	
Lined channels	
a. Asphalt	0.013–0.017
b. Brick	0.012–0.018
c. Concrete	0.011–0.020
d. Rubble or riprap	0.020–0.035
e. Vegetal	0.030–0.40[b]
Excavated or dredged	
Earth, straight and uniform	0.020–0.030
Earth, winding, fairly uniform	0.025–0.040
Rock	0.030–0.045
Unmaintained	0.050–0.14
Natural channels (minor streams, top width at flood stage < 100 ft)	
Fairly regular section	0.03–0.07
Irregular section with pools	0.04–0.10

[a]Dimensional units contained in numerical term in formula.
[b]See References 2, 5, 16. (Vanes with depth and velocity.)
Note: 1 in. = 2.54 cm; 1 ft = 0.305 m.

nels (Chow 1959; FHWA 1961; SCS 1954) and also for natural channels and floodplains (Chow 1959). Equation 6-16 can be rewritten as:

$$S_f = \left(\frac{Qn}{1.49AR^{2/3}}\right)^2 \tag{6-31a}$$

or, in SI units:

$$S_f = \left(\frac{Qn}{AR^{2/3}}\right)^2 \tag{6-31b}$$

and

$$S_f = \frac{29.1n^2}{R^{4/3}}\left(\frac{V^2}{2g}\right) \tag{6-32a}$$

or, in SI units:

$$S_f = \frac{19.85n^2}{R^{4/3}}\left(\frac{V^2}{2g}\right) \tag{6-32b}$$

For the 24-inch concrete sewer discussed in the preceding section, the friction slope by Manning's equation would be computed from Equation 6-31 as:

$$S_f = \left(\frac{25 \times 0.012}{1.49 \times 3.1416 \times 0.5^{2/3}}\right)^2 = 0.0104 \text{ feet/foot}$$

or by (6-32) as:

$$S_f = \frac{29.1 \times (0.012)^2}{0.5^{4/3}}\left(\frac{7.96^2}{64.4}\right) = 0.0104 \text{ feet/foot}$$

This produces a friction head loss, H_f, of 1.04 feet for 100 feet of sewer, which is about 16% higher than H_f computed from the Hazen-Williams equation. This difference is attributed to the difference between the Manning n value and C_{HW} in the Hazen-Williams formula.

In view of the importance of analyzing normal depth and other hydraulic properties in partially-full circular conduits, a hydraulic element chart is presented in Figure 6.13. The ratios Q/Q_f, V/V_f, A/A_f, R/R_f, and n/n_f are called the hydraulic elements. A symbol without subscript represents the value of a variable when the conduit is flowing partially full and the subscript "f" represents values for the full conduit with the same slope of the energy grade line. Figure 6.13 is a hydraulic-

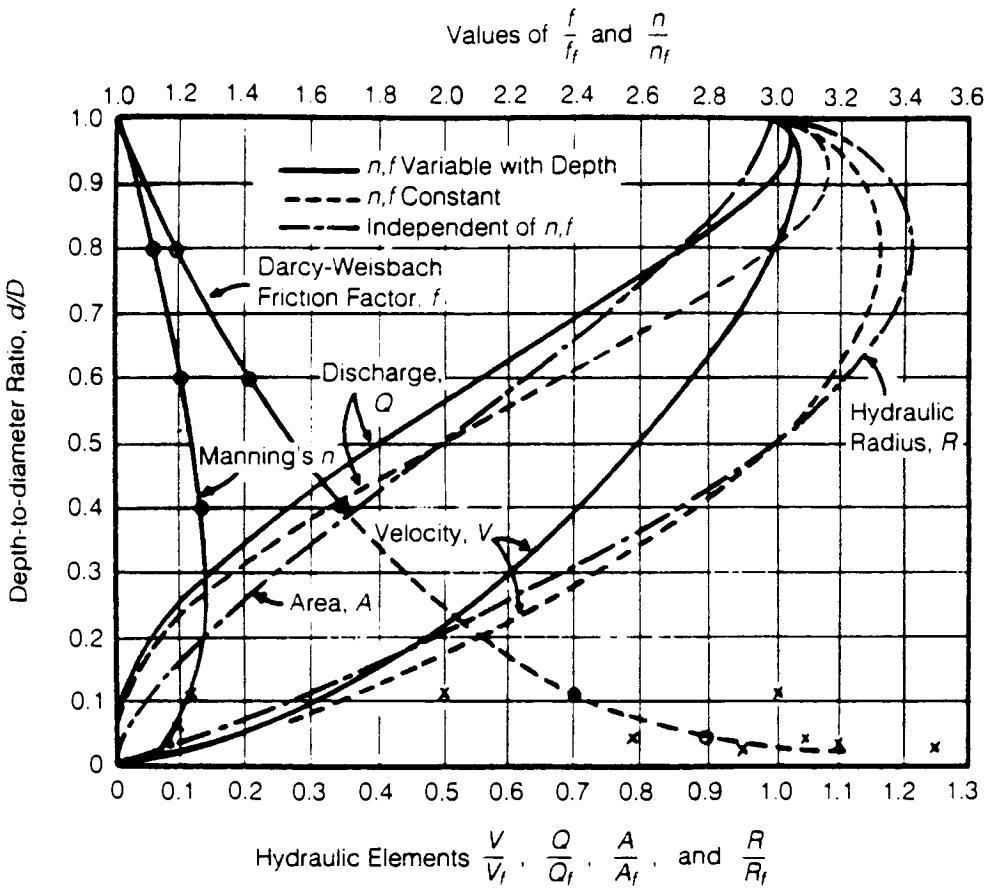


Figure 6.13—Hydraulic elements graph for partially filled circular sewers (ASCE, 1982).

elements graph for circular conduits with uniform roughness throughout the surface area. These curves are shown in tabular form in Table 6.6. Geometric relationships for various types of flow sections are listed in Table 6.7. Note the apparent variation in Manning's n with depth in Figure 6.13. This problem is avoided in an alternative method for normal depth proposed by Christensen (1984) for flow in partially full conduits.

X. MINOR LOSSES

In addition to the friction loss along a channel or conduit, there is usually a local loss of energy associated with any sudden change due to transitions, junctions, bends, entrances, exits, obstructions, and control devices, such as orifices and gates. These losses occur over a relatively short distance and are usually represented by a steep slope or sudden drop in the energy grade line.

TABLE 6.6. Tabular Values of Hydraulic Elements of Pipes (n constant) (ASCE 1972).

$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{Q_n}{D^{8/3}S^{1/2}}$	$\frac{Q_c}{D^{5/2}}$	$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{Q_n}{D^{8/3}S^{1/2}}$	$\frac{Q_c}{D^{5/2}}$
0.01	0.0013	0.00007	0.0006	0.51	0.4027	0.239	1.4494
0.02	0.0037	0.00031	0.0025	0.52	0.4127	0.247	1.5041
0.03	0.0069	0.00074	0.0055	0.53	0.4227	0.255	1.5598
0.04	0.0105	0.00138	0.0098	0.54	0.4327	0.263	1.6166
0.05	0.0147	0.00222	0.0153	0.55	0.4426	0.271	1.6741
0.06	0.0192	0.00328	0.0220	0.56	0.4526	0.279	1.7328
0.07	0.0242	0.00455	0.0298	0.57	0.4625	0.287	1.7924
0.08	0.0294	0.00604	0.0389	0.58	0.4724	0.295	1.8531
0.09	0.0350	0.00775	0.0491	0.59	0.4822	0.303	1.9147
0.10	0.0409	0.00967	0.0605	0.60	0.4920	0.311	1.9773
0.11	0.0470	0.01181	0.0731	0.61	0.5018	0.319	2.0410
0.12	0.0534	0.01417	0.0868	0.62	0.5115	0.327	2.1058
0.13	0.0600	0.01674	0.1016	0.63	0.5212	0.335	2.1717
0.14	0.0668	0.01952	0.1176	0.64	0.5308	0.343	2.2886
0.15	0.0739	0.0225	0.1347	0.65	0.5404	0.350	2.3068
0.16	0.0811	0.0257	0.1530	0.66	0.5499	0.358	2.3760
0.17	0.0885	0.0291	0.1724	0.67	0.5594	0.366	2.4465
0.18	0.0961	0.0327	0.1928	0.68	0.5687	0.373	2.5182
0.19	0.1039	0.0365	0.2144	0.69	0.5780	0.380	2.5912
0.20	0.1118	0.0406	0.2371	0.70	0.5872	0.388	2.6656
0.21	0.1199	0.0448	0.2609	0.71	0.5964	0.395	2.7416
0.22	0.1281	0.0492	0.2857	0.72	0.6054	0.402	2.8188
0.23	0.1365	0.0537	0.3116	0.73	0.6143	0.409	2.8977
0.24	0.1449	0.0585	0.3386	0.74	0.6231	0.416	2.9783
0.25	0.1535	0.0634	0.3667	0.75	0.6319	0.422	3.0606
0.26	0.1623	0.0686	0.3957	0.76	0.6405	0.429	3.1450
0.27	0.1711	0.0739	0.4259	0.77	0.6489	0.435	3.2314
0.28	0.1800	0.0793	0.4571	0.78	0.6573	0.441	3.3200
0.29	0.1890	0.0849	0.4893	0.79	0.6655	0.447	3.4111
0.30	0.1982	0.0907	0.5226	0.80	0.6736	0.453	3.5051
0.31	0.2074	0.0966	0.5969	0.81	0.6815	0.458	3.6020
0.32	0.2167	0.1027	0.5921	0.82	0.6893	0.463	3.7021
0.33	0.2260	0.1089	0.6284	0.83	0.6969	0.468	3.8062
0.34	0.2355	0.1153	0.6657	0.84	0.7043	0.473	3.9144
0.35	0.2450	0.1218	0.7040	0.85	0.7115	0.477	4.0276
0.36	0.2546	0.1284	0.7433	0.86	0.7186	0.481	4.1466
0.37	0.2642	0.1351	0.7836	0.87	0.7254	0.485	4.2722
0.38	0.2739	0.1420	0.8249	0.88	0.7320	0.488	4.4057
0.39	0.2836	0.1490	0.8672	0.89	0.7384	0.491	4.5486

TABLE 6.6. Continued

$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{Q_n}{D^{8/3}S^{1/2}}$	$\frac{Q_c}{D^{5/2}}$	$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{Q_n}{D^{8/3}S^{1/2}}$	$\frac{Q_c}{D^{5/2}}$
0.40	0.2934	0.1561	0.9104	0.90	0.7445	0.494	4.7033
0.41	0.3032	0.1633	0.9546	0.91	0.7504	0.496	4.8724
0.42	0.3130	0.1705	0.9997	0.92	0.7560	0.497	5.0602
0.43	0.3229	0.1779	1.0459	0.93	0.7612	0.498	5.2727
0.44	0.3328	0.1854	1.0929	0.94	0.7662	0.498	5.5182
0.45	0.3428	0.1929	0.1410	0.95	0.7707	0.498	5.8119
0.46	0.3527	0.201	1.1900	0.96	0.7749	0.496	6.1785
0.47	0.3627	0.208	1.2400	0.97	0.7785	0.494	6.6695
0.48	0.3727	0.216	1.2908	0.98	0.7817	0.489	7.4063
0.49	0.3827	0.224	1.3427	0.99	0.7841	0.483	8.8261
0.50	0.3927	0.232	1.3956	1.00	0.7854	0.463	—

In long conduits where $L/D >> 1000$ these local losses are usually very small in comparison to the friction losses and the minor losses can be neglected. However, if the channel or conduit is very short and/or there are a number of manholes, changes in direction, junctions, or changes in pipe size then the sum of these losses can exceed the friction loss.

In terms of calculations, the loss is expressed either as a coefficient times the velocity head or as a coefficient times the difference in velocity heads, depending on the type of loss involved. This is usually written:

$$H_L = K_c \left(\frac{V^2}{2g}\right) \tag{6-33}$$

in which H_L is the minor head loss, K_c is a loss coefficient dependent on the type of loss, and $V^2/2g$ is the velocity head.

The loss coefficient and the form of the equation are different depending on the type of loss, whether flow is open channel or pressure flow, and at times whether flow is subcritical or supercritical. Full discussion and values of coefficients are given in several references (Chow 1959; Brater and King 1976; Rouse 1961; Hendrickson 1964; USBR 1977; FHWA 1978; FHWA 1985; NBS 1938; Bowers 1950). The following are useful minor head loss formulas for hydraulic structures commonly found in storm sewer systems and open channels.

A. Transition Losses

A transition is a location where a conduit or channel changes size. The change in cross-sectional area results in a change in velocity, which

means a loss of head. The energy losses in contraction or expansion can be expressed in terms of the kinetic energy at the two ends of the transition as shown in Equations (6-34) and (6-35). K_c (contraction) and K_e (expansion) are the transition loss coefficients. Table 6.8 gives some typical values. V_1 is the upstream velocity above the transition and V_2 is the downstream velocity below the transition.
Contraction:

$$H_c = K_c \left(\frac{V_2^2}{2g} - \frac{V_1^2}{2g} \right) \text{ for } V_2 > V_1 \tag{6-34}$$

Expansion:

$$H_e = K_e \left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g} \right) \text{ for } V_1 > V_2 \tag{6-35}$$

Henderson (1969) recommends using $H_t = K_t (V_1 - V_2)^2/2g$ (where H_t = transition loss coefficient) in place of 6-34 and 6-35, but indicates the two methods give very similar results when $1.5 < V_1/V_2) < 2.5$. For pipes with pressure flow, the loss coefficients listed in Tables 6.9 and 6.10 can be used in conjunction with Equation 6-33 for sudden and gradual expansions, respectively. For sudden contractions in pipes with pressure flow, the loss coefficients listed in Table 6.11 can be used in conjunction with Equation 6-33, in which K_c is replaced by K_3. For values of K_2 and K_3 outside the ranges found in Tables 6.10 and 6.11, see Daily and Harleman (1966). As will be noted from these tables and from the discussion on junctions below, the designer is advised to design transitions carefully to minimize losses.

B. Entrance Losses

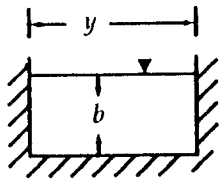
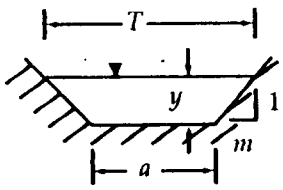
Entrance losses to box culverts and pipes of various materials can be estimated by using the entrance loss coefficients listed in Table 6.12 in conjunction with Equation 6-33. See also the detailed discussion in Chapter 8.

C. Manhole and Junction Losses

Junctions are locations where two or more pipes join together to form another pipe or channel. They represent another critical point in a storm drainage system that must be designed as a transition through which the flow is changing direction.

Multiple pipes or channels coming together at a junction should flow together smoothly to avoid high head losses. Items that promote turbulent flow and high losses include a large angle between the two (>60°), a large vertical difference between the two (greater than 6 inches between the two inverts), and absence of a semicircular channel or

TABLE 6.7. Geometric Relationships for Various Flow Sections.

Section	Area (A)	Wetted Perimeter (P)	Hydraulic Radius (R)	Top Width (T)
Rectangular 	by	$b + 2y$	$\frac{by}{b + 2y}$	T
Trapezoidal 	$(a + my)y$	$a + 2y\sqrt{1 + m^2}$	$\frac{(a + my)y}{a + 2y\sqrt{1 + m^2}}$	$a + 2my$

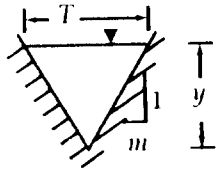
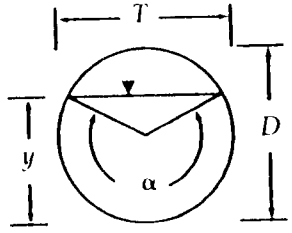
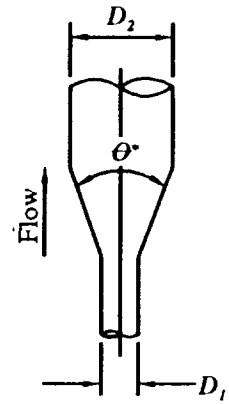
Triangular 	my^2	$2y\sqrt{1+m^2}$	$\dfrac{my}{2\sqrt{1+m^2}}$	$2my$
Circular  $\alpha = 2\sin^{-1}\left[\dfrac{2\sqrt{y(D-y)}}{D}\right]$	$(\alpha - \sin\alpha)D^2/8$	$\dfrac{\alpha D}{2}$	$\dfrac{D}{4}\left[1 - \dfrac{\sin\alpha}{\alpha}\right]$	$(\sin {}^1/_2\,\alpha)D$ or $2\sqrt{y(D-y)}$

TABLE 6.8. Storm Sewer Energy Loss Coefficient. (expansion, contraction) (Linsley and Franzini 1964)

(a) Expansion (K_e)

θ*	$\frac{D_2}{D_1} = 3$	$\frac{D_2}{D_1} = 1.5$
10	0.17	0.17
20	0.40	0.40
45	0.86	1.06
60	1.02	1.21
90	1.06	1.14
120	1.04	1.07
180	1.00	1.00

*The angle θ is the angle in degrees between the sides of the tapering section.



(b) Pipe Entrance from Reservoir

Bell-mouth	$H_L = 0.04 \frac{V2}{2g}$
Square-edge	$H_L = 0.5 \frac{V2}{2g}$
Groove end U/S For Concrete Pipe	$H_L = 0.2 \frac{V2}{2g}$

(c) Contractions (K_c)

$\frac{D_2}{D_1}$	K_c
0	0.5
0.4	0.4
0.6	0.3
0.8	0.1
1.0	0

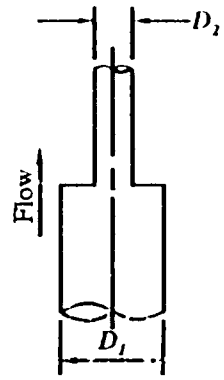


TABLE 6.9. Values of K_e for Determining Loss of Head Due to Sudden Enlargement in Pipes, from the Formula $H_2 = K_2(V_1^2/2g)$ (AISI 1985)

d_2/d_1 = ratio of larger pipe to smaller pipe V_1 = velocity in smaller pipe

$\frac{d_2}{d_1}$	Velocity, V_1, in feet per second												
	2	3	4	5	6	7	8	10	12	15	20	30	40
1.2	.11	.10	.10	.10	.10	.10	.10	.09	.09	.09	.09	.09	.08
1.4	.26	.26	.25	.24	.24	.24	.24	.23	.23	.22	.22	.21	.20
1.6	.40	.39	.38	.37	.37	.36	.36	.35	.35	.34	.33	.32	.32
1.8	.51	.49	.48	.47	.47	.46	.46	.45	.44	.43	.42	.41	.40
2.0	.60	.58	.56	.55	.55	.54	.53	.52	.52	.51	.50	.48	.47
2.5	.74	.72	.70	.69	.68	.67	.66	.65	.64	.63	.62	.60	.58
3.0	.83	.80	.78	.77	.76	.75	.74	.73	.72	.70	.69	.67	.65
4.0	.92	.89	.87	.85	.84	.83	.82	.80	.79	.78	.76	.74	.72
5.0	.96	.93	.91	.89	.88	.87	.86	.84	.83	.82	.80	.77	.75
10.0	1.00	.99	.96	.95	.93	.92	.91	.89	.88	.86	.84	.82	.80
∞	1.00	1.00	.98	.96	.95	.94	.93	.91	.90	.88	.86	.83	.81

TABLE 6.10. Values of K_2 for Determining Loss of Head Due to Gradual Enlargement in Pipes from the Formula $H_2 = K_2(V_1^2/2g)$ (AISI 1985)

d_2/d_1 = ratio of diameter of larger pipe to diameter of smaller pipe. Angle of cone is twice the angle between the axis of the cone and its side.

$\frac{d_2}{d_1}$	Angle of cone													
	2°	4°	6°	8°	10°	15°	20°	25°	30°	35°	40°	45°	50°	60°
1.1	.01	.01	.01	.02	.03	.05	.10	.13	.16	.18	.19	.20	.21	.23
1.2	.02	.02	.02	.03	.04	.09	.16	.21	.25	.29	.31	.33	.35	.37
1.4	.02	.03	.03	.04	.06	.12	.23	.30	.36	.41	.44	.47	.50	.53
1.6	.03	.03	.04	.05	.07	.14	.26	.35	.42	.47	.51	.54	.57	.61
1.8	.03	.04	.04	.05	.07	.15	.28	.37	.44	.50	.54	.58	.61	.65
2.0	.03	.04	.04	.05	.07	.16	.29	.38	.46	.52	.56	.60	.63	.68
2.5	.03	.04	.04	.05	.08	.16	.30	.39	.48	.54	.58	.62	.65	.70
3.0	.03	.04	.04	.05	.08	.16	.31	.40	.48	.55	.59	.63	.66	.71
∞	.03	.04	.05	.06	.08	.16	.31	.40	.49	.56	.60	64	.67	.72

R0021001

TABLE 6.11. Values of K_3 for Determining Loss of Head Due to Sudden Contraction from the Formula $H_3 = K_3(V_2^2/2g$ (AISI 1985)

d_2/d_1 = ratio of larger to smaller diameter V_2 = velocity in smaller pipe

$\frac{d_2}{d_1}$	Velocity, V_2, in feet per second												
	2	3	4	5	6	7	8	10	12	15	20	30	40
1.1	.03	.04	.04	.04	.04	.04	.04	.04	.04	.04	.05	.05	.06
1.2	.07	.07	.07	.07	.07	.07	.07	.08	.08	.08	.09	.10	.11
1.4	.17	.17	.17	.17	.17	.17	.17	.18	.18	.18	.18	.19	.20
1.6	.26	.26	.26	.26	.26	.26	.26	.26	.26	.25	.25	.25	.24
1.8	.34	.34	.34	.34	.34	.34	.33	.33	.32	.32	.31	.29	.27
2.0	.38	.38	.37	.37	.37	.37	.36	.36	.35	.34	.33	.31	.29
2.2	.40	.40	.40	.39	.39	.39	.39	.38	.37	.37	.35	.33	.30
2.5	.42	.42	.42	.41	.41	.41	.40	.40	.39	.38	.37	.34	.31
3.0	.44	.44	.44	.43	.43	.43	.42	.42	.41	.40	.39	.36	.33
4.0	.47	.46	.46	.46	.45	.45	.45	.44	.43	.42	.41	.37	.34
5.0	.48	.48	.47	.47	.47	.46	.46	.45	.45	.44	.42	.38	.35
10.0	.49	.48	.48	.48	.48	.47	.47	.46	.46	.45	.43	.40	.36
∞	.49	.49	.48	.48	.48	.47	.47	.47	.46	.45	.44	.41	.38

R0021002

TABLE 6.12. Entrance Loss Coefficents for Culverts (FHWA 1985) Outlet Control, Full or Partly Full Entrance Head Loss.

$$H_e = k_e \left(\frac{V^2}{2g}\right)$$

Type of *Structure and Design of Entrance*	*Coefficent* k_e
Pipe, Concrete	
Projecting from fill, socket end (groove-end)	0.2
Projecting from fill, sq. cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded (radius = 1/12D)	0.2
Mitered to conform to fill slope	0.7
*End-section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° levels	0.2
Side- or slope-tapered inlet	0.2
Pine, or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headwal)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
*End-section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side- or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 barrel dimension, or beveled edges on 3 sides	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of 1/2 barrel dimension, or beveled top edge	0.2
Wingwall at 10° to 25° to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side- or slope-tapered inlet	0.2

*Note: "End-section conforming to fill slope," made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both *inlet* and *outlet* control. Some end sections, incorporating a *closed* taper in their design have a superior hydraulic performance.

benching at the bottom of the junction box in the case of the pipes. Special problems arise when smaller pipes join a larger one at a junction.

Losses at sewer junction manholes can typically account for 20–30% of total head losses, though wide variances are possible. In extreme cases, junction manholes can account for much higher percentages of losses. These losses can be minimized by careful design and construction. For a complete discussion, see Marsalek (1985, 1986, 1987).

In a straight-through manhole where there is no change in pipe size, the minor loss can be estimated by:

$$H_m = 0.05\left(\frac{V^2}{2g}\right) \tag{6-36}$$

Junction losses for other configurations in closed conduits can be estimated from the equations shown in Figures 6.14 and 6.15 (City of Austin 1987), and Figure 6.16 (AISI 1985) (see also discussion of losses at manholes in Chapter 8).

D. Bend Losses

Bend losses in open channels can be estimated by using the bend loss coefficients listed in Table 6.13 in conjunction with Equation (6-33). If the ratio of the radius of the bend, r, to the width of the channel, b, is equal to or greater than 3.0, the loss is negligible.

Bend losses in closed conduits can be estimated by using the bend loss coefficients shown in Figures 6.15 (in conjunction with Equation 6-33) and 6.16. These values are for the high Reynolds numbers usually encountered in hydraulic engineering practice.

XI. CALCULATION OF WATER SURFACE PROFILES

The prediction of drawdown and backwater curves such as those shown in Figure 6.9 is essential for the design of storm sewers and open channels. In the case of drawdown curves, it is sometimes possible to achieve a substantial savings in cost by reducing the wall height or by reducing the size of the conduit, thereby also avoiding overhead structures.

Two stepwise calculation methods are available: the Direct Step Method, which yields the **location** of chosen depths; and the Standard Step Method giving **depths** at selected locations. Both methods provide information necessary for plotting the nonuniform water surface profile. The Direct Step Method is recommended for hand calculations under prismatic channel/pipe conditions since it does not involve tedious iterative operations. The Standard Step Method on the other hand, is iterative and is best handled by a computer.

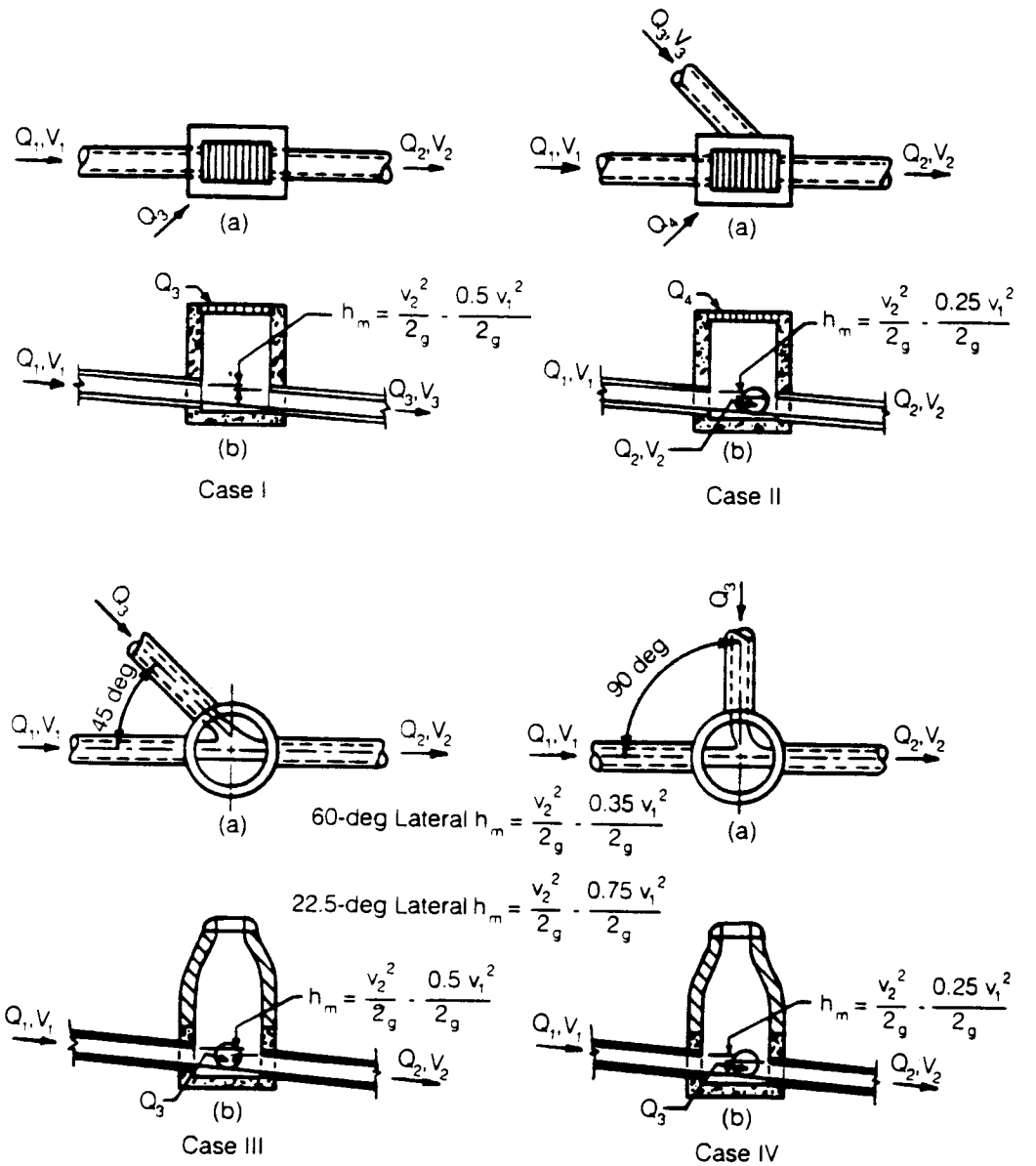
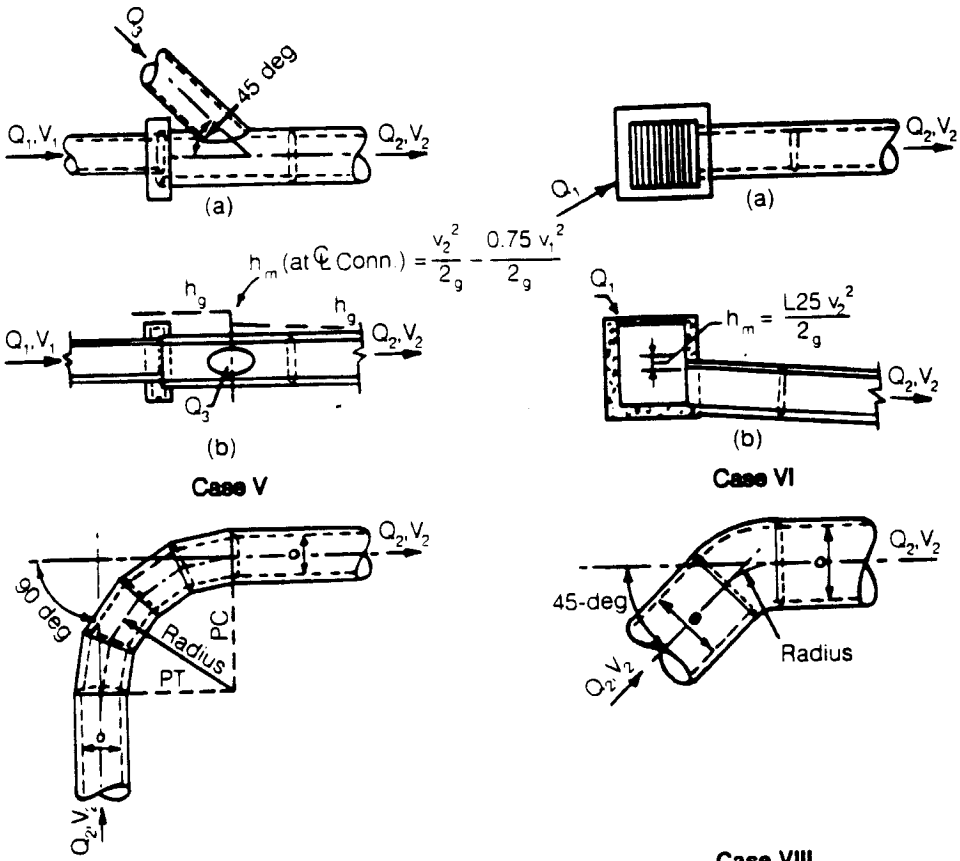


Figure 6.14—Minor head losses due to turbulence at structures: case I—inlet on main line (a) plan and (b) section, case II—inlet on main line with branch lateral (a) plan and (b) section, case III—manhole on main line with 45-deg branch lateral (a) plan and (b) section, and case IV—manhole on main line with 90-deg branch lateral (a) plan and (b) section (City of Austin, 1987).

As noted in previous sections, all calculations of water surface profiles must begin at a control section where the depth is known. They will proceed in the direction of flow if the flow is supercritical, or in the upstream direction if the flow is subcritical. Numerous references are available for step-wise and analytical calculations (Chow 1959; Von



Case VII

Head loss applied at PC for length of curve.

Radius = diam. of pipe $h_m = 0.50 \frac{v_2^2}{2g}$

Radius = (2-8) diam. of pipe $h_m = 0.25 \frac{v_2^2}{2g}$

Radius = (8-20) diam. of pipe $h_m = 0.40 \frac{v_2^2}{2g}$

Radius = greater than 20 diam. of pipe $h_m = 0$

When curves other than 90 deg are used, apply the following factors to 90-deg curves:
60-deg curve 85%,
45-deg curve 70%, and
22.5-deg curve 40%.

Case VIII

Head loss applied at beginning of bend

90-deg bend $h_m = 0.50 \frac{v_2^2}{2g}$

60-deg bend $h_m = 0.43 \frac{v_2^2}{2g}$

45-deg bend $h_m = 0.35 \frac{v_2^2}{2g}$

22.5-deg bend $h_m = 0.20 \frac{v_2^2}{2g}$

Figure 6.15—Minor head losses due to turbulence at structures: case V—45-deg Wye connection or cut in (a) plan and (b) section, case VI—inlet or manhole at beginning of line (a) plan and (b) section, case VII—conduit on 90-deg curves, and case VIII—bends where radius is equal to diameter of pipe (City of Austin, 1987).

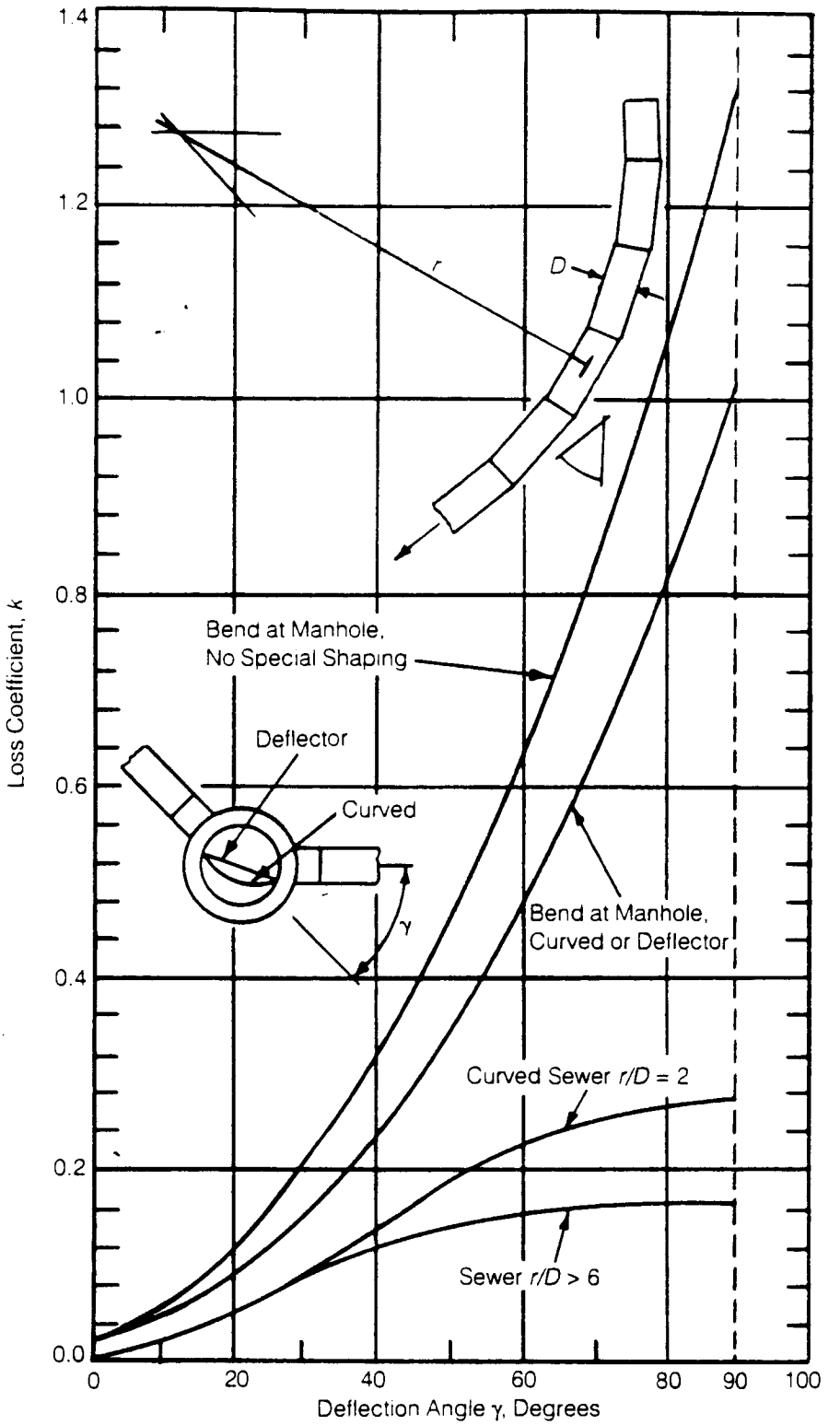


Figure 6.16—Sewer bend loss coefficient (AISI, 1985).

TABLE 6.13. Bend Loss Coefficients in Open Channels.

r/b (1)	k_b (2)
2.5	0.02
2.0	0.07
1.5	0.12
1.0	0.25

Seggern 1950; Kiefer and Chu 1955; Chow 1955) as well as for graphical water surface profile computations (Chow 1959; Thomas 1934).

The Direct Step Method for calculating the length of prismatic channel or conduit between cross-sections where the water surface elevation is known is based on the following equation:

$$L = \frac{E_2 - E_1}{S_o - \bar{S}_f} = \frac{(d + h_v)_2 - (d + h_v)_1}{S_o - \bar{S}_f} \tag{6-37}$$

where

L = length between cross-sections.
E = sum of depth and velocity heads (specific energy).
d = depth.
h_v = velocity head.
$\bar{S}_f$ = average friction slope between the two cross-sections.
S_o = bed slope of the channel or conduit.

Examples of both the direct-step and standard-step methods are illustrated below.

Example 6-4: Direct Step Method Calculation of M-1 Backwater Curve An 8-foot diameter circular storm sewer is laid at a slope of 0.001 ft per ft and is conveying a flow of 80 cfs. At its confluence with an open channel, the depth of flow is 8.0 ft. Assume a roughness coefficient of 0.013 when flowing full. Critical depth is 2.20 ft and normal depth is 3.24 ft. Determine the length required to reach normal depth. The calculations are shown in Table 6.14, adapted from ASCE (1972).

Example 6-5: Standard Step Calculation of S-2 Drawdown Curve Flow from a channel on a mild slope enters a 36-inch circular pipe which is laid on a slope of 0.02 ft per ft. Determine the water surface profile downstream from the pipe entrance for a discharge of 40 cfs. Assume a roughness coefficient of 0.013, which is constant with depth. Critical depth is 2.06 feet and normal depth is 1.36 ft. Since $d_c > d_n$, the uniform flow is supercritical, so calculations should start at critical depth and proceed downstream.

TABLE 6.14. Calculation of M-1 Backwater Curve by Direct-Step Method (ASCE 1972).

d (ft) (1)	d/D (2)	Q/Q_f (3)	Q_f (cfs) (4)	V_f (fps) (5)	V/V_f (6)	V (fps) (7)	h_v (ft) (8)	$d + h_v$ (ft) (9)	S (10)	S_c (11)	$S_c - S_o$ (12)	$\Delta(d + h_v)$ (ft) (13)	ΔL (ft) (14)
8.00	1.00	1.00	80	1.59	1.00	1.59	0.04	8.04	0.85×10^{-4}				
										1.1×10^{-4}	-8.9×10^{-4}	-1.98	2,220
6.00	0.75	0.79	101	2.01	0.98	1.97	0.06	6.06	1.35×10^{-4}				
										1.7×10^{-4}	-8.3×10^{-4}	-0.97	1,170
5.00	0.62	0.60	133	2.64	0.90	2.38	0.09	5.09	2.1×10^{-4}				
										3.4×10^{-4}	-6.6×10^{-4}	-0.93	1,410
4.00	0.50	0.40	200	3.97	0.80	3.18	0.16	4.16	4.8×10^{-4}				
										6.0×10^{-4}	-4.0×10^{-4}	-0.36	900
3.60	0.45	0.33	242	4.80	0.75	3.60	0.20	3.80	7.2×10^{-4}				
										8.6×10^{-4}	-1.4×10^{-4}	-0.29	2,080
3.24	0.40	0.28	290	5.77	0.72	4.16	0.27	3.51	10.0×10^{-4}				
													$L = 7{,}780$

Explanation:

Col. 1: Assumed depths between initial depth of 3.24 ft and terminal depth of 8.00 ft.
Col. 2: Col. 1 ÷ D (diameter of sewer).
Col. 3: From Fig. 6-13 for d/D in Col. 2.
Col. 4: 80 cfs ÷ Col. 3.
Col. 5: Col. 4 ÷ 50.3 sq ft.
Col. 6: From Fig. 6-13 for d/D in Col. 2.
Col. 7: Col. 5 × Col. 6.
Col. 8: Velocity head for Col. 7.
Col. 9: Col. 1 + Col. 8.
Col. 10: S from Manning equation (6-17) for D = 8 ft, Q_f of Col. 4, and n = 0.013.
Col. 11: Arithmetic mean of successive pairs in Col. 10.
Col. 12: Col. 11 − S_o.
Col. 13: Difference between successive amounts in Col. 9.
Col. 14: Col. 13 ÷ Col. 12.

Note 1: To obtain accuracy, the assumed depths should be chosen such that differences between successive values of velocities shown in Col. 7 will be less than 10 percent; to make Table 6.14 concise, this limit was not met.
Note 2. Ft × 0.3048 = m; cfs × 1.7 = cu m/min.

Arbitrary lengths of reach and assumed depths of flow are selected and S_f is obtained by averaging successive S_f values. The friction loss is applied to the previous H and the result is compared with the assumed value. If they are sufficiently close, the calculation proceeds to the next station. If not, a new trial depth of flow is selected for the current station and the process repeated until the assumed and computed water surface elevations agree, within some specified tolerance. The calculations are shown in Table 6.15, adapted from ASCE (1972).

XII. SPECIAL HYDRAULIC STRUCTURES

A. Storm Sewer Inlets

Storm sewer inlets are the means by which urban runoff enters the sewer system. A storm sewer system is usually designed on the assumption of full-flowing pipes, often with little regard for how surface runoff is delivered to it. In fact, the storm sewer inlet is an important element of the design in its own right. As shown in Figure 6.17, stormwater inlets can take many forms, but usually are classified as curb inlets, gutter inlets, or slotted drains. No one inlet is best suited for all conditions. The hydraulics of flow into an inlet are based on principles of weir and orifice flow, modified by laboratory and field observation of entrance losses under controlled conditions.

Curb inlets are installed along street sections having curbs and gutters to intercept stormwater runoff and to allow its passage into a storm sewer. Inlets can be located at low points (sumps), directly upstream from street intersections, and at intermediate locations as well. The spacing of these intermediate curb inlets depends on several criteria but is usually controlled by rate of flow and the permissible water spread toward the street crown. The type of road is also important since the greater the speed and volume of traffic, the greater the potential for hydroplaning. On the other hand, it is also considered acceptable practice to allow some periodic and temporary flooding of low volume streets.

Given the maximum allowable width of street flooding, the designer can compute the allowable rate of runoff in the curb section represented as a flat triangular channel by:

$$Q = 0.56 \left(\frac{Z}{n}\right) d^{8/3} S^{1/2} \tag{6-38a}$$

or, in SI units:

$$Q = 0.38 \left(\frac{Z}{n}\right) d^{8/3} S^{1/2} \tag{6-38b}$$

TABLE 6.14. Calculation of M-1 Backwater Curve by Direct-Step Method (ASCE 1972).

d (ft) (1)	d/D (2)	Q/Q_f (3)	Q_f (cfs) (4)	V_f (fps) (5)	V/V_f (6)	V (fps) (7)	h_v (ft) (8)	$d + h_v$ (ft) (9)	S (10)	S_e (11)	$S_e - S_o$ (12)	$\Delta(d + h_v)$ (ft) (13)	ΔL (ft) (14)
8.00	1.00	1.00	80	1.59	1.00	1.59	0.04	8.04	0.85×10^{-4}	1.1×10^{-4}	-8.9×10^{-4}	−1.98	2,220
6.00	0.75	0.79	101	2.01	0.98	1.97	0.06	6.06	1.35×10^{-4}	1.7×10^{-4}	-8.3×10^{-4}	−0.97	1,170
5.00	0.62	0.60	133	2.64	0.90	2.38	0.09	5.09	2.1×10^{-4}	3.4×10^{-4}	-6.6×10^{-4}	−0.93	1,410
4.00	0.50	0.40	200	3.97	0.80	3.18	0.16	4.16	4.8×10^{-4}	6.0×10^{-4}	-4.0×10^{-4}	−0.36	900
3.60	0.45	0.33	242	4.80	0.75	3.60	0.20	3.80	7.2×10^{-4}	8.6×10^{-4}	-1.4×10^{-4}	−0.29	2,080
3.24	0.40	0.28	290	5.77	0.72	4.16	0.27	3.51	10.0×10^{-4}				$L = 7{,}780$

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Col. 11: Arithmetic mean of successive pairs in Col. 10.
Col. 12: Col. 11 − S_o.
Col. 13: Difference between successive amounts in Col. 9.
Col. 14: Col. 13 ÷ Col. 12.

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Note 2. Ft × 0.3048 = m; cfs × 1.7 = cu m/min.

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or, in SI units:

$$Q = 0.38 \left(\frac{Z}{n}\right) d^{8/3} S^{1/2} \tag{6-38b}$$

TABLE 6.15. Calculation of S–2 Drawdown Curve by Standard-Step Method (ASCE 1982).

Q (cfs) (1)	$Qn/D^{8/3}$ (2)	Sta. (3)	Elev Water Surface (ft) (4)	Elev Invert (ft) (5)	d (ft) (6)	d/D (7)	A (sq ft) (8)	V (fps) (9)	$V^2/2g$ (ft) (10)	Assume Elev H (ft) (11)	$Qn/D^{8/3}S^{1/2}$ for d/D (12)	S_f (13)	Avg S_f (14)	H_f (15)	Computed Elev H (16)
40	.02778	0 + 00	102.06	100.00	2.06	0.687	5.18	7.72	0.93	102.99	0.378	.0054	—	—	102.99
—	—	0 + 10	101.54	99.80	1.74	0.580	4.25	9.41	1.37	102.91	0.295	.0089	.0072	0.07	102.92
—	—	0 + 30	100.99	99.40	1.59	0.530	3.80	10.53	1.72	102.71	0.255	.0119	.0104	0.21	102.71
—	—	0 + 60	100.29	98.80	1.49	0.497	3.50	11.43	2.03	102.32	0.230	.0146	.0133	0.40	102.31
—	—	1 + 50	98.39	97.00	1.39	0.463	3.20	12.50	2.43	100.82	0.203	.0187	.0167	1.50	100.81
—	—	2 + 50	96.37	95.00	1.37	0.457	3.14	12.74	2.52	98.89	0.199	.0195	.0191	1.91	98.90

Explanation:

Col. 2: Useful constant for reach calculations.

Col. 3: Stationing arbitrarily established to define backwater curve.

Col. 4: First line of calculation is known elevation at point of control, remaining lines assumed elevations.

Col. 6: Depth of flow, Col. 4 − Col. 5.

Col. 8: Area of channel for depth in Col. 6, from Table 6.6 A/D^2 values for d/D in Col. 7.

Col. 9: Q/A, Col. 1 ÷ Col. 8.

Col. 10: $V^2/2g$ from Col. 9.

Col. 11: Col. 4 + Col. 10.

Col. 12: From Table 6.6 $Qn/D^{8/3}S^{1/2}$ for d/D in Col. 7.

Col. 13: (Col. 2 ÷ Col. 12)^2.

Col. 14: Average friction slope of adjacent stations (no entry on first line of reach).

Col. 15: Length between stations × Col. 14.

Col. 16: First line for actual elevation of energy head at point of control. Remaining lines are values at prior station plus or minus Col. 15. Col. 16 should be in approximate agreement with Col. 11 before proceeding.

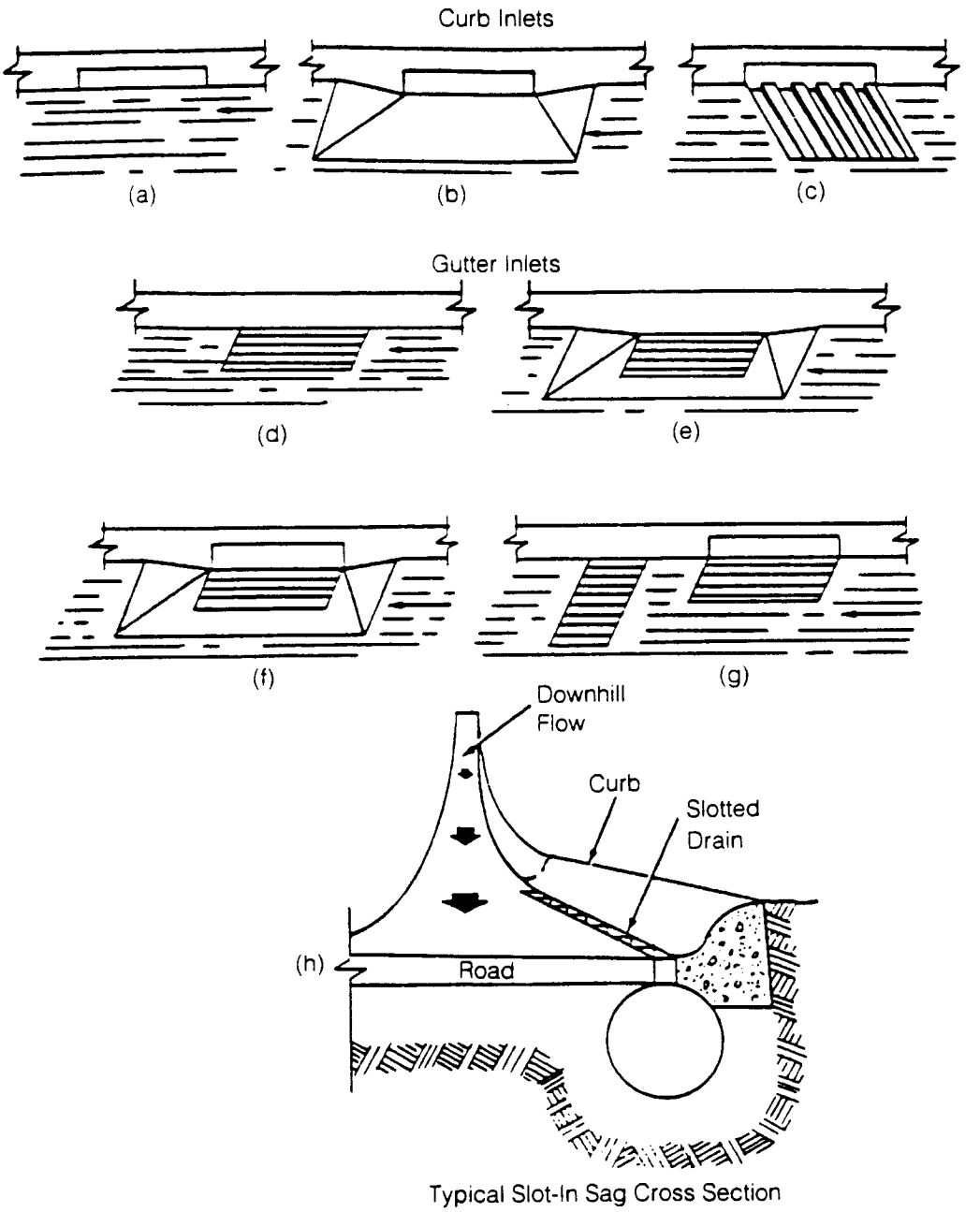

Figure 6.17—Stormwater inlets: (a) undepressed, (b) depressed, (c) deflector inlet, (d) undepressed, (e) depressed, (f) combination inlet—grate placed directly in front of curb opening depressed, (g) multiple inlet undepressed, and (h) slotted drain (AISI, 1985).

Equation (6-38) is derived from Manning's equation where Z is the reciprocal of the street cross slope ($1/S_x$), n is Manning's roughness coefficient, S is the longitudinal slope of the street, and d is the depth of flow at the curb, and Zd (used in the nomograph) is the width of water spread. A nomograph is presented in Figure 6.18 for Equation

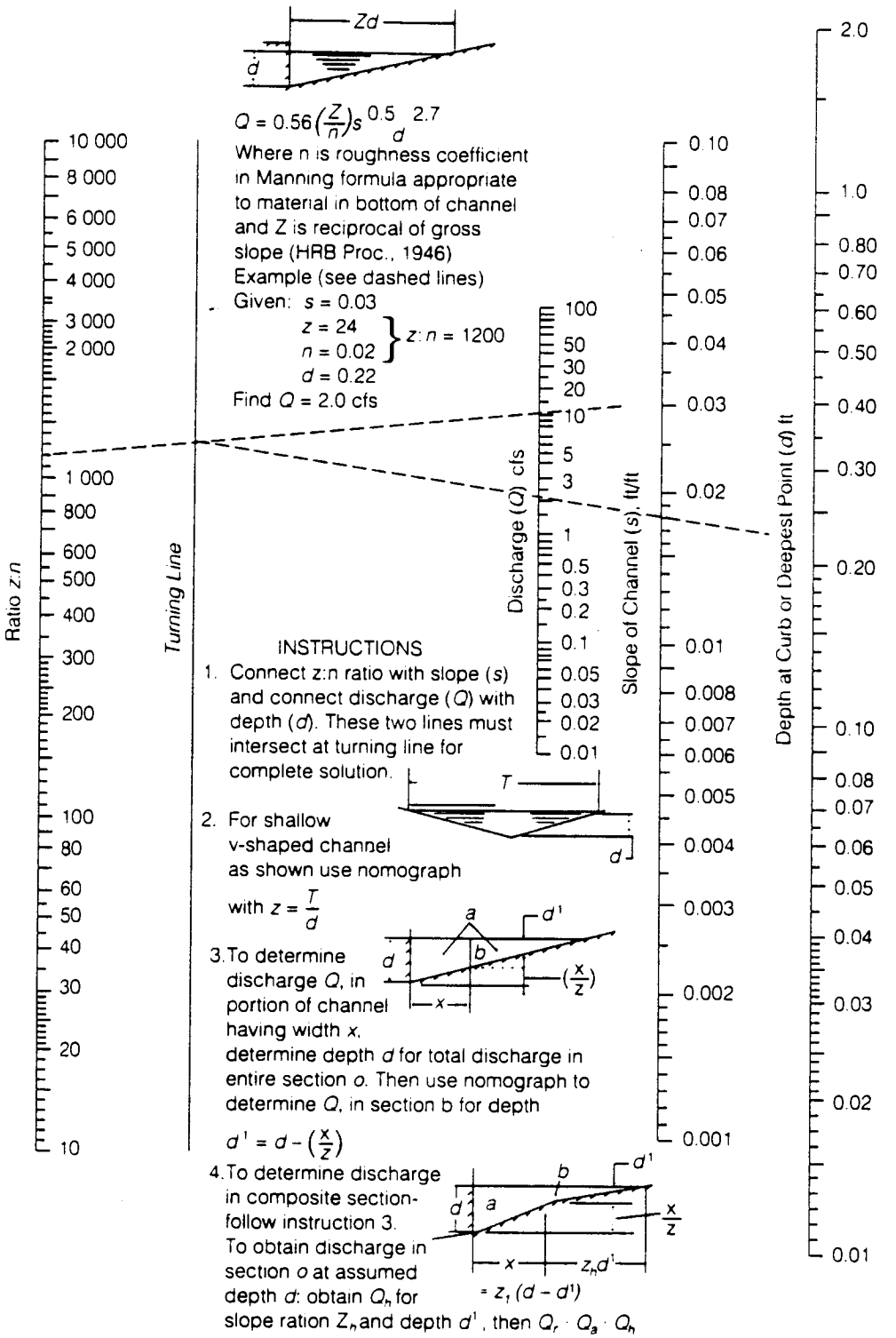

Figure 6.18—Nomograph for flow in triangular channels (AISI, 1985).

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6-38. Note that the Manning equation was modified in deriving equation (6-38) because the hydraulic radius does not adequately describe the gutter cross-section, particularly where the top width Zd may be more than 40 times the depth at the curb.

If the actual runoff exceeds the allowable flooding, then the inlets need to be redesigned, placed closer together, or made larger. The capacity of each individual inlet depends on whether the inlet is located on a continuous grade or in a sump. The capacity also depends on the reduction factor used for blockage, the interception ratio for the total flow, whether the inlet throat is depressed, and whether deflectors are used.

Actual capacity is determined from nomographs or equations found in state highway department manuals, textbooks, and in some regional specifications. Two very good references are the Denver Regional Urban Storm Drainage Criteria Manual (1969) and the Federal Highway Administration report on Drainage of Highway Pavements (1984).

As a practical matter, curb inlets are notoriously inefficient and there never can be too many. The final design should show roughly one 4-foot wide inlet for every 3 cfs on a street with longitudinal slope of 2% or less.

Grate inlets are flush-mounted inlets installed along drainageways and streets for the purpose of intercepting storm water and directing it into the underground storm drainage system. Grate inlets can be sized large enough to drain an area quickly, which in turn will reduce infiltration into sanitary sewers and minimize freeze-thaw cycle effects under nearby pavements. Conversely, grate inlets can be undersized to reduce outflow and intentionally back-up storm water into a detention basin which will reduce peak rates of flow downstream.

Grate inlets function best when located in a sump. For depths of water not exceeding 0.4 feet, the weir equation is used:

$$Q = C_w P d^{3/2} \tag{6-39}$$

where $C_w = 3.0$ (1.66 in SI units), P is the perimeter of the grate opening assuming no bars, and d is the depth of flow above the grate. If the grate is adjacent to a curb, that side of the grate is not counted in the perimeter. For depths of flow exceeding 1.4 feet, the orifice equation is used:

$$Q = C_o A (2gh)^{1/2} \tag{6-40}$$

where $C_o = 0.6$, A is the total area of opening, g is the acceleration due to gravity, and h is the head above the center of the orifice. For depths greater than 0.4 feet but less than 1.4 feet, the capacity will be somewhere between those calculated by the weir and orifice equations.

The minimum length of clear opening parallel to the direction of flow necessary to allow the flow to fall through the opening and clear the downstream end of the bars can be estimated by the equation:

$$L = 0.5\ V(t + d)^{1/2} \tag{6-41a}$$

where L is the minimum length of clear opening in feet, V is the average velocity of the approach water, d is the approaching water depth in feet, and t is the grate thickness in feet.

In SI units:

$$L = 0.91\ V(t + d)^{1/2} \tag{6-41b}$$

Again, reference is made to FHWA (1984) for detailed capacity calculations.

B. Culverts

Culverts are defined as short conduits sized to pass a stream or drainage swale under a roadway or railway embankment without overtopping. Culverts can be circular, elliptical, arched, rectangular, or square in cross-section. They are frequently mounted in a headwall with an improved entrance. The headwall may have multiple barrels if allowable headwater is limited. Under heavy runoff conditions, the culvert inlet can restrict the amount of water passing into the culvert, and the culvert is said to flow under inlet control. The culvert capacity is given by the orifice equation, as modified by a projecting or non-projecting entrance.

In cases where the culvert is long and there is tailwater submergence, the culvert can flow under outlet control. Here the capacity is dependent on the culvert length, slope, roughness, and entrance and exit loss, as well as the tailwater depth. The hydraulics of flow through culverts under outlet control are based on the energy equation, plus the Manning equation to represent the friction slope. Culvert design charts for both inlet and outlet control are abundant and are commonly used in an iterative design process. Design criteria include size and cross-section of the culvert, culvert type, amount of headwater available, and the details of the entrance. The best known charts are those by the Federal Highway Administration (1985). The hydraulic design of culverts is discussed further in Chapter 8.

C. Energy Dissipators

Energy dissipators are used in a storm drainage system to reduce flow velocity (kinematic energy) and thereby reduce pipe scouring or stream bank and bed erosion. Depending on the type of channel or pipe lining, flow velocities may become excessive well below the critical velocity.

Open channel or non-pressurized flow velocities in a conduit or channel can be estimated by Manning's equation. For culverts under inlet control (partially full) velocities can be estimated from Manning's equa-

tion or by dividing the rate of flow by the cross-sectional area under outlet control (full flow).

Allowable flow velocities vary with conditions. For example, according to the Virginia Erosion and Sediment Control Handbook, (Virginia Soil and Water Conservation Commission 1982), permissible velocities for earth-lined channels range from 2.5 feet per second for loamy sand and sandy loam soils to 6.0 feet per second for coarse gravel, shale, and hard pan. Permissible velocities for grass-lined channels range from 2.5 feet per second for red fescue and red top grasses to 6.0 feet per second for Bermuda grass on soils that are not highly erodible. According to the Virginia Department of Highways and Transportation Drainage Manual, the allowable velocity in riprapped channels ranges from 12 feet per second for 50–150 pound stone to about 25 feet per second for 3–10-ton stone.

There are countless types of energy dissipators, including riprap, poured in place concrete, drop overs (which reduce channel slopes), and manufactured devices. Even grass linings on channels and natural undergrowth can be considered as energy dissipators since they act to retard flow velocity, though their effectiveness is limited under high depth and velocity conditions.

The effectiveness of a dissipator varies by type and magnitude of storm. However, it should be emphasized that certain practices produce poor energy dissipation and should be avoided. For example, straw bales, silt fences, and small stone will not only prove worthless, but they will have to be cleaned up somewhere downstream following a severe storm.

D. Drop Structures

Drop structures are used in open channels as a means of reducing channel gradient, and thereby channel velocity. Thus, drops can be considered a form of energy dissipator.

Reduced channel velocity means less capacity for a given cross-section. Other disadvantages of a vertical drop include the turbulence and erosive effect of the falling water on the drop structure, necessitating high maintenance. Because of these disadvantages, vertical drops are not recommended and sloping drops are used instead. As with vertical drops, stream banks in the vicinity of slope drops must be protected from erosion and scouring. The hydraulics of drop structures are complex, and reference is made to various publications by the USDA Soil Conservation Service for design specifications.

E. Outlet Structures

The purpose of an outlet structure in a storm drainage system is to deliver storm water that has been collected in a conduit, channel, or detention basin to a downstream receiving channel or body of water. Four factors influence the design of outlet structures, of which adequate

capacity is the first and most obvious. A constricting outlet structure inadequate to carry the design flow will cause flooding and possible damage upstream. On the other hand, an oversized outlet structure is a waste of funds since peak flows will pass through unattenuated. In cases such as detention basins, only a predetermined amount of flow through an outlet can be permitted. Usually multiple stages will be required to maintain outflows at pre-development levels across the full range of design flows.

Second, non-erosive velocities must be maintained, recognizing that inadequate velocity will produce sediment deposits within the drainage way. This of course will cause increased maintenance and/or reduced capacities, which together can combine to increase the potential for flooding and related property damage.

XIII. ROUTING

With recent emphasis on watershed-wide control of stormwater runoff in many states, the essence of stormwater management planning has become the analysis of individual subarea runoff impacts at a downstream point of interest under changing land use in the watershed. This concept requires that runoff control facilities be sized and located in such a way that not only local runoff control is achieved, but also that flow targets at downstream points are met. Thus, drainage engineers must be able to generate subarea hydrographs as well as route these discharge hydrographs through the system to identify those subareas best suited for various runoff control facilities. The flow routing operation is critical to the analysis and to the identification of sewer/channel elements susceptible to surcharge and street flooding.

Routing of unsteady discharge is essential to the analysis of subarea timing effects and also to the design of storm drainage facilities, such as detention basins and pumping stations. Inputs to the routing sequence will be outflow hydrographs from individual subareas. The routing operation consists of computing the movement and attenuation of an inflow hydrograph as it passes through the storm sewer system, or, under surcharge conditions, through both the storm sewer and overland system. Flow routing may involve anything from a simple time shift based on time-of-travel to solution of the Saint-Venant or gradually varied unsteady flow equations, taking into account backwater and pressure flow conditions. In both cases, we are computing the discharge hydrograph (or stage hydrograph) at the downstream end of a pipe, channel, or detention basin and accounting mathematically for the effects of storage on flow through the element.

A. Types of Routing Methods

There are numerous methods for routing flows through a sewer, channel or storage basin in an urban drainage system. All methods can

be classified as either hydraulic (based on Saint-Venant equations) or hydrologic (based on mass continuity plus a storage-outflow relation). The intent here is to identify those most commonly used and provide references for the detailed computational procedures. The simplest routing method is the time-of-travel shift whereby the discharge or even the entire inflow hydrograph is translated without attenuation to the downstream end of the flow element.

The travel time is defined simply as the element length divided by the full-flow velocity estimated by the Manning or the Hazen-Williams equation. This has become a satisfactory routing technique for relatively simple systems without significant backwater effects created by pipe looping, diversion structures, or submerged outfalls. It is particularly well-suited to the separate storm drainage system where storage is insufficient to cause significant peak reduction.

Flow routing in an open channel on the other hand can have substantial peak attenuation because of the storage in overbank areas during flood stage. This should be treated by one of the hydrologic routing methods such as the Muskingum method or the kinematic wave technique to simulate important properties of the flood wave. Neither of these methods has the ability to represent backwater conditions emanating from a downstream location, however.

The Muskingum method is a well-documented hydrologic procedure for channel routing and can be found in any standard reference on engineering hydrology. It simply combines the equation of mass continuity with a two-parameter storage-inflow-outflow relation for the channel reach to obtain a value of the outflow hydrograph at the end of a specified time step. The routing calculations continue until the entire inflow hydrograph has been routed through the reach.

The kinematic wave routing method was developed originally for overland flow on a plane surface and has also been applied to flow routing in pipes and channels. The method derives from the assumption that gravity and friction forces dominate all other terms in the momentum equation for gradually varied unsteady flow and, therefore, that the bed slope of the channel or pipe equals the friction slope. This allows use of a uniform flow equation in combination with mass conservation to obtain a solution for the outflow hydrograph leaving a pipe or channel segment.

Theoretically, the kinematic wave method cannot attenuate the flood wave unless kinematic shock is incorporated in the computations. Nor can this method be used when severe backwater conditions are present, since it is based on tracking disturbances that move only downstream. Despite these disadvantages, the kinematic wave routing method is widely used.

A modified form that permits attenuation of the peak has been developed by the Soil Conservation Service (1983) in its hydrologic model TR-20. Similarly, the kinematic wave method is used to simulate urban drainage systems in HEC-1, developed by the U.S. Army Corps of

Engineers Hydrologic Engineering Center. It also serves as the principal routing technique in the urban drainage model known as DR3M, developed by the U.S. Geological Survey Urban Studies Program (Alley and Smith 1982), and is used in the Transport block of SWMM. The reader is referred to a text on kinematic wave modeling by Stephenson and Meadows (1986) for a full discussion of numerical methods and comparisons between time-shift, Muskingum, and kinematic wave routing methods.

Routing of inflow hydrographs through flood control reservoirs and detention basins is performed by storage-indication or modified Puls methods. These two are computationally equivalent and involve the construction of an elevation-storage-outflow relationship which can be used with the mass continuity equation to obtain the outflow hydrograph. An example of modified Puls routing is presented later in this chapter.

The routing of combined sewer flows in a complex network involving looped pipes, diversion structures, and submerged outfalls usually requires solving the one-dimensional unsteady, gradually varied flow equations presented previously in this chapter. Under these conditions, the flow is controlled by acceleration terms, in addition to gravity and friction forces, and consequently the kinematic wave method is no longer applicable.

The methods available for routing combined sewer flows are generally referred to as full dynamic methods and require the use of complex finite difference schemes working at small time intervals for computational stability. While numerous computer models are available, the best known in the United States is the Extended Transport model (EXTRAN) in the SWMM program. EXTRAN and other hydraulic transport models in its class have the capability to represent complex backwater conditions caused by flow diversion structures, sewer looping, tidal outfalls, flow reversals, and pressure or surcharge flow.

EXTRAN can also handle a variety of sewer configurations including parallel pipes and trapezoidal channels. EXTRAN uses an explicit form of the Saint-Venant equations particularly sensitive to time step size and prone to numerical instability. In general, all models in this class, while affording complete representation of all elements in a complex sewer system, require considerable hydraulic modeling and computing expertise. Further information on hydraulic and hydrologic routing models is contained in Chapter 7.

B. Detention Basin Routing By Modified Puls Method

1. Basic Equations

The Modified Puls method consists of repetitive solutions of the continuity equation and is based on the assumptions that (1) the reservoir water surface remains horizontal; and (2) the outflow from the

reservoir is a unique function of storage. This method is sometimes called the storage-indication method. The methods are numerically equivalent and can be applied to major flood control projects as well as to small detention basins.

The continuity equation may be expressed as:

$$\bar{I} - \bar{O} = \frac{\Delta S}{\Delta t} \tag{6-42}$$

where

$\bar{I}$ = mean inflow into reservoir during routing period Δt,
$\bar{O}$ = mean outflow from reservoir during routing period Δt,
ΔS = change in reservoir storage during routing period Δt.

Equation 6-42 may be expressed by:

$$\frac{I_1 + I_2}{2} - \frac{O_1 + O_2}{2} = \frac{S_2 - S_1}{\Delta t} \tag{6-43}$$

where subscripts 1 and 2 denote the beginning and end, respectively, of a routing period, Δt.

The assumption implicit in Equation 6-43 is that the discharge varies linearly with time during a routing period Δt. This assumption must be borne in mind when selecting a routing period.

Equation 6-43 may be re-stated as follows:

$$(I_1 + I_2) + \left(\frac{2S_i}{\Delta t} - O_1\right) = \frac{2S_2}{\Delta t} + O_2 \tag{6-44}$$

In Equation 6-44, all terms on the left-hand side are known from preceding routing computations. The terms on the right-hand side involving S_2 and O_2 are unknown and must be determined by storage routing.

2. *General Routing/Design Procedure for Stormwater Detention Facilities*

Subtract the pre-development runoff volume from the post-development volume to arrive at an estimation of trial detention basin size. The difference represents the approximate storage requirement. Then refer to storage-elevation data for the site to estimate the headwater available. Next, refer to culvert design charts by FHWA for inlet/outlet control to determine the approximate size of outlet required to pass maximum allowable outflow at a headwater depth corresponding to the storage requirement. A full routing analysis of the trial basin/outlet is based on the following steps in the Modified Puls method:

(a) Compute pre-development hydrograph from which the maximum basin outflow is determined.

(b) Compute post-development hydrograph which is to be routed through the proposed detention basin.
(c) Compute a table and/or curve of water depth versus storage (a function of basin geometry); water depth is measured above the basin floor or lowest outflow pipe invert elevation.
(d) Compute a table and/or curve of water depth versus outflow, recognizing that stage-discharge relationships are a function of the particular outlet structure and whether it is flowing under inlet or outlet control.
(e) Select a routing period Δt such that there are 5 or 6 points on the rising side of the inflow hydrograph, one of which coincides with the inflow peak.
(f) Construct a graph of $[(2S/\Delta t) + O]$ versus O where
S = storage volume, cfs-hours
Δt = routing period, hours
O = outflow rate, cfs
(g) A routing procedure is now initiated using a tabular method for the solution of Modified Puls equation in equation (6-44).
(h) Compare the maximum outflow rate with the allowable rate of discharge from the drainage area.
(i) Adjust size, shape, and/or outlet structure if the maximum routed outflow is greater than the allowable.
(j) Repeat the entire routing procedure for alternative designs.

Example 6-6: Single-Stage Detention Basin Routing and Design The example shown in Figure 6.19 (Kibler 1988) is a 150-acre watershed located in central Pennsylvania. The developed condition in Figure 6.19 includes a 53-acre site in the middle of the watershed which will be developed as a detached multi-family town house project. In addition, the area to the right will be developed as single family homes. The pre-development land use was corn/hay agricultural in the town house area. The detention basin adjacent to the roadway embankment is to be designed as a single-stage structure which will control the Q_{25} post-development runoff peak at the pre-development level. The pre- and post-development hydrographs were computed by the tabular hydrograph method in SCS Technical Release 55 (SCS 1986) and are shown in Figure 6.20 (Kibler 1988).

Based on the TR55 tabular hydrographs in Figure 6.21, the storage volume is estimated at approximately 6 acre-feet from Fig. 6.1, TR55 (SCS 1986). The pond depth and headwater at this storage level will be about 8.0 feet from Figure 6.21. With 8.0 feet of head, a 48-inch CMP culvert will discharge 125 cfs under inlet control, as shown in Figure 6.22. Since this is close to the target outflow of 113 cfs, try a single 48-inch CMP outlet. The following steps constitute the detention routing procedure (FHWA 1985):

(a) The pre- and post-development hydrographs are given in Figure 6.21. The problem is to size the detention basin and outlet works to reduce the post-development flood peak of 186 cfs to the pre-development level of 113 cfs.

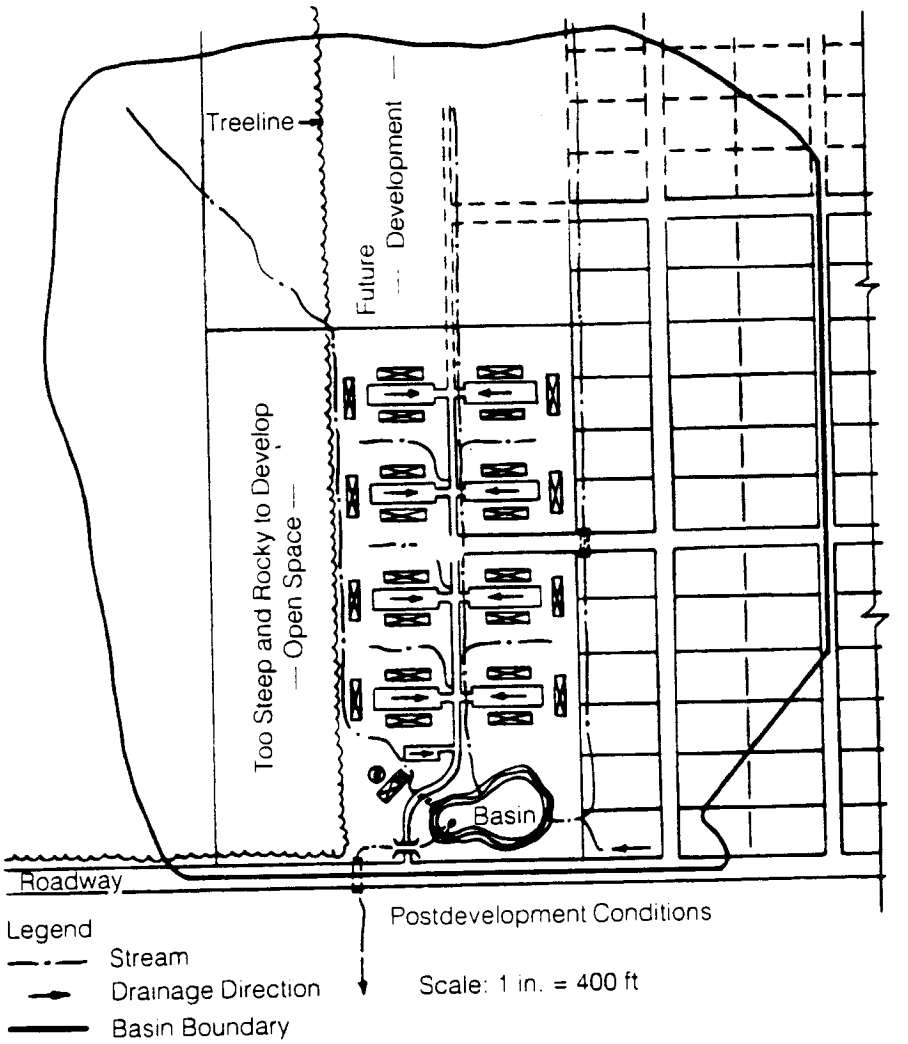

Figure 6.19—Post-development conditions in 150-acre watershed used in detention basin routing example.

(b) The storage-elevation curve for the proposed site of the detention facility is shown in Figure 6.21. The outlet invert is assumed to be at elevation 1060.0 feet for dry-pond alternative.

(c) The hydraulic performance curve for a 48-inch CMP culvert from FHWA HDS No. 5 (1985) with projecting inlet flowing under inlet control is shown in Figure 6.22. This is an assumed size and flow condition.

(d) Select the routing interval Δt such that there are 5 or 6 points on the rising limb of the inflow hydrograph, one of which coincides with the inflow peak. From Figure 6.21, Δt = 10 min.

(e) Construct table and graph of $[(2S/\Delta t) + O]$ versus outflow using Figures 6.21 and 6.22 with Δt = 10 minutes. These are shown in Table 6.16 and Figure 6.23.

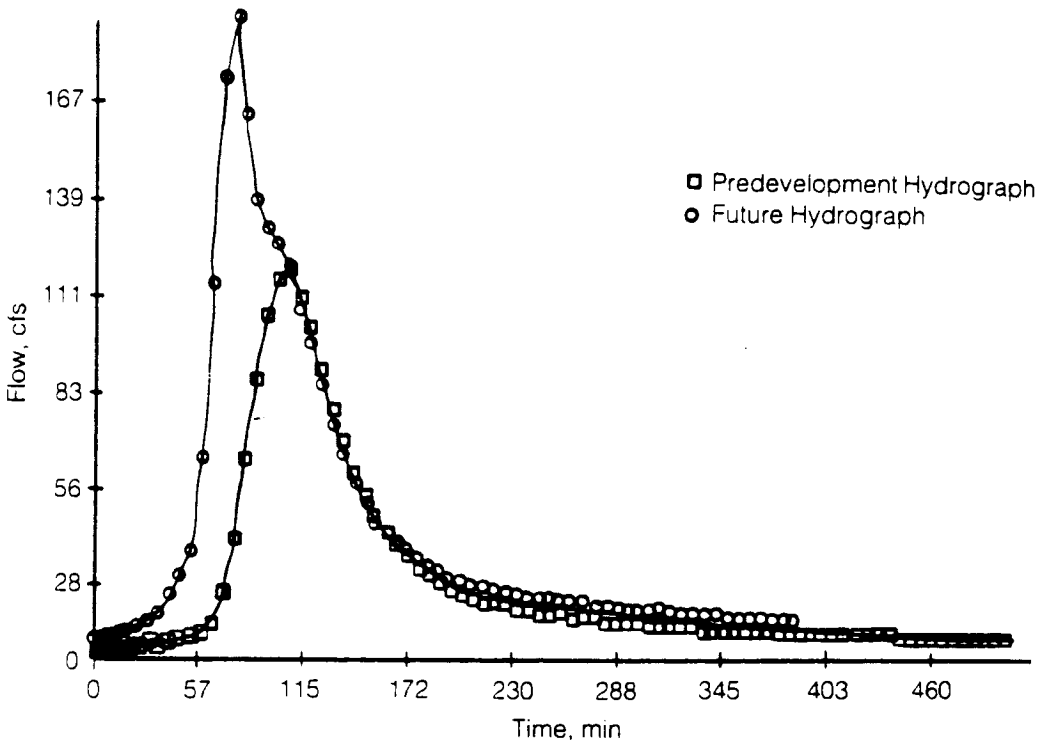

Figure 6.20—Q_{25} hydrographs by SCS tabular method for existing and developed conditions in 150-ac watershed (Kibler, 1988).

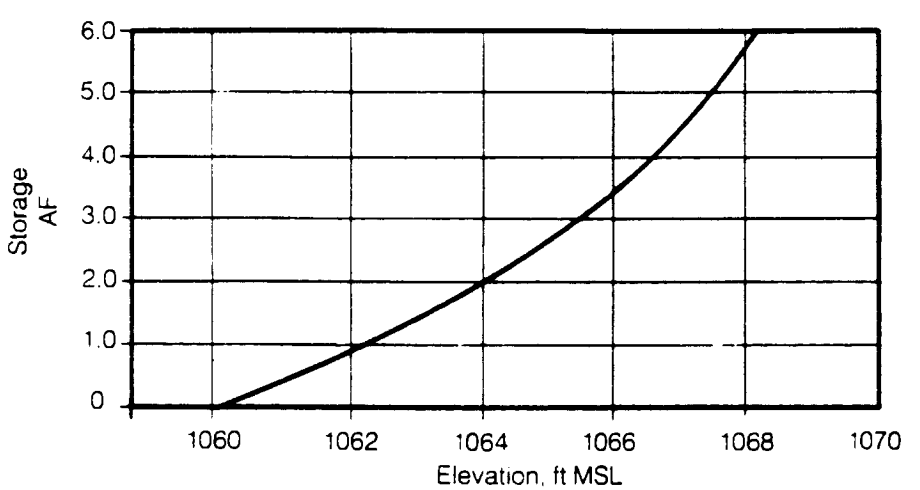

Figure 6.21—Storage-elevation curve for proposed detention site (ft × 0.304 8 = m).

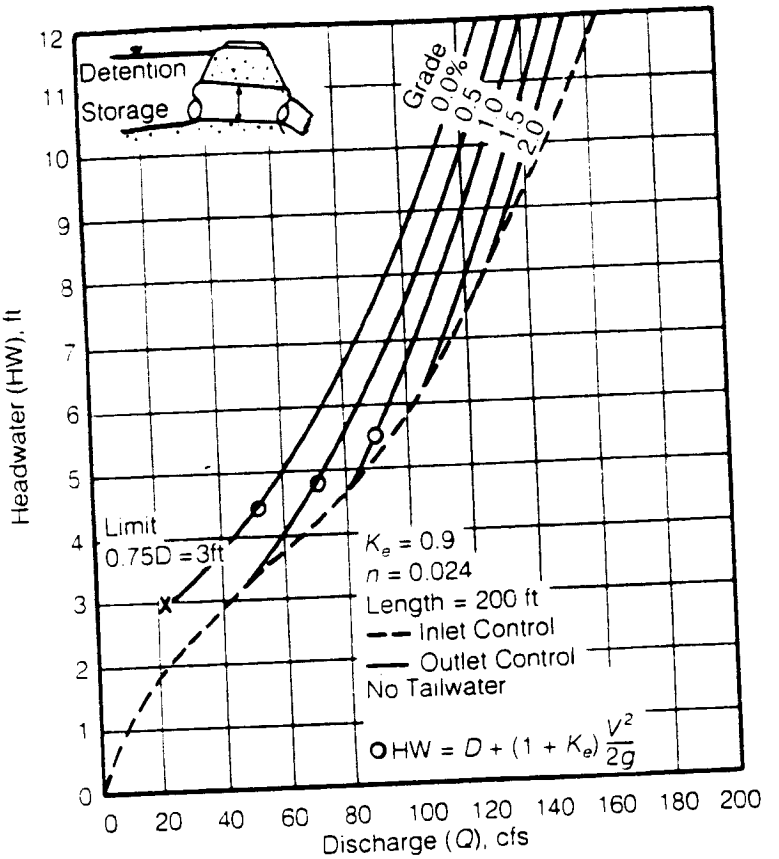

Figure 6.22—Discharge performance curves for 48-in. cmp culvert (cfs × 0.028 32 = m³/s, ft × 0.304 8 = m, and in. × 25.4 = mm) (FHWA, 1985).

TABLE 6.16. $2S/\Delta t$ for 48-inch CM Pipe Culvert with Δt = 10 Minutes

Elev (1)	Storage, AF (2)	Storage, cfs-hrs (3)	O cfs (4)	$2S/\Delta t + O$ (5)
1060	0.00	0.0	0	0
1061	0.40	4.8	6	63
1062	1.00	12.0	21	165
1063	1.50	18.0	40	256
1064	2.00	24.0	65	353
1065	2.60	31.2	87	461
1066	3.40	40.8	103	593
1067	4.20	50.4	115	720
1068	5.40	64.8	125	903
1069	7.20	86.4	135	1172
1070	10.00	120.0	147	1587

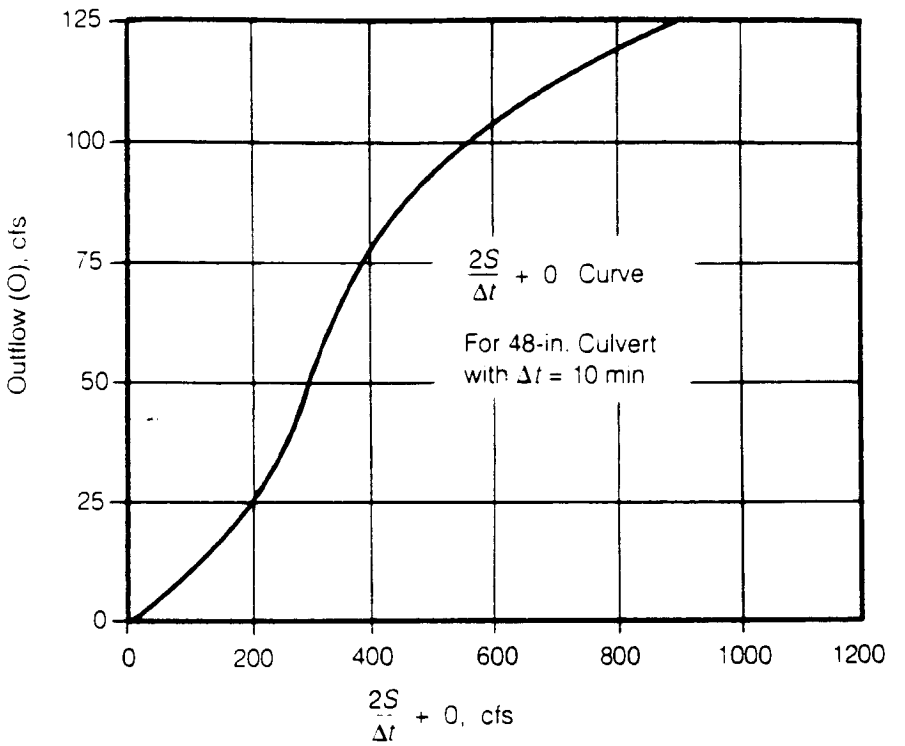

Figure 6.23—Plot of outflow versus ($^{2S}/\Delta t$ + 0) for 48-in. culvert. (cfs × 0.028 32 = m ^{3}/s and in. × 25.4 = mm).

(f) Construct a routing table for solution of equation (6-43), using the post-development inflow hydrograph in Figure 6.20 and the $2S/\Delta t$ plot in Figure 6.23. The completed routing table is shown in Table 6.17.

(g) Compare the maximum outflow rate, cfs, with the allowable rate of 113 cfs. Since this 48-inch CMP design results in a peak outflow rate of 113 cfs it is probably acceptable without further modification. However, other options should be investigated as necessary. Note that TR55 produces a storage estimate which is about 38% higher than that by Modified Puls routing. The TR55 approximate routing procedure is very useful for preliminary sizing purposes, but should always be followed by a full routing analysis to determine design adequacy.

3. *Multi-stage Detention Facilities*

Because of the need to control multiple storms with return periods usually between 2–50 years, the engineer is faced with the challenge of sizing multiple outlets in a detention facility. These outlets take the form of various orifices, weirs, and drop inlets which will control detention basin releases at pre-development levels in the watershed. Multiple openings are usually mounted in a riser box that discharges to an outlet pipe or culvert. The design of the total structure is complicated by: (1) submergence of the inlets by water in the riser; and (2) shifting control between inlet and outlet control in the outfall pipe. An inter-

TABLE 6.17. Routing table and detention storage analysis for 48-inch pipe.

Time Hrs:min (1)	I_1 cfs (2)	$I_2 + I_2$ cfs (3)	$2S/\Delta t - O$ cfs (4)	$2S/\Delta t + O$ cfs (5)	O_2 cfs (6)
11:00	7	15	0	0	0
11:10	8	18	13	15	1
11:20	10	22	25	31	3
11:30	12	30	39	47	4
11:40	18	46	57	69	6
11:50	28	87	81	103	11
12:00	59	207	126	168	21
12:10	148	334	217	333	58
12:20	186	320	359	551	96
12:30	134	256	463	679	108
12:40	122	232	495	719	112
12:50	110	201	501	727	113
13:00	91	163	480	702	111
13:10	72	129	433	643	105
13:20	57	103	370	562	96
13:30	46	84	305	473	84
13:40	38	71	255	389	67
13:50	33	62	222	326	52
14:00	29	55	202	284	41
14:10	26	49	187	257	35
14:20	23	44	74	136	31
14:30	21	41	64	118	27
14:40	20	39	57	105	24
14:50	19	37	48	96	22
15:00	18	33	43	85	21

Notes: (1) Max. storage occurs at max. outflow at time 12:50. Since $2S/\Delta t + O = 727$ cfs at this time, $S = 51.17$ cfs-hrs $= 4.3$ AF. From storage-elevation curve, this produces a maximum depth of 6.8 feet at elevation 1066.8.

(2) The TR55 (1986) estimate of required storage is based on the following calculations:

$Q_o/Q_i = 113/186 = 0.61$

$V_s/V_r = 0.23$ from Fig. 6.1 TR55

V_s (AF) $= (0.23 \times 2.07 \text{ inches} \times 150 \text{ acres})/12$

$= 5.96$ AF (about 38% high)

active computer solution is usually required. Chamberlain (1985) has developed a micro-computer program capable of analyzing up to 9 stages with 10 different outlet types including: circular and rectangular orfices, V-notch weir, proportional weir, rectangular weir, perforated riser, drop inlet with grate; emergency spillway; and circular discharge pipe. This multi-stage program has been documented by Kibler and Seybert (1989).

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Chapter 7
COMPUTER MODELING

I. INTRODUCTION

Computer modeling became an integral part of storm drainage planning and design in the mid-1970s. Several federal agencies undertook major software developments, including The Hydrologic Engineering Center (HEC) of the U.S. Army Corps of Engineers with the Storage, Treatment and Overflow Runoff Model (STORM) and the HEC-1 and HEC-2 series; the U.S. Soil Conservation Service with TR-20 and WSP2; and the U.S. Environmental Protection Agency, who sponsored development of the Storm Water Management Model (SWMM). These were soon supplemented by a plethora of proprietary models, many of which were simply variants on the originals. The proliferation of microcomputers in the 1980s has made it possible for virtually every engineer to use state-of-the-art analytical technology for purposes ranging from analysis of individual pipes to comprehensive stormwater management plans for entire cities.

In addition to the simulation of hydrologic and hydraulic processes, models can have other uses. They can provide a quantitative means to test alternatives and controls before implementation of expensive measures in the field. If a model has been calibrated and verified at a minimum one site, it may be used to simulate non-monitored conditions and to extrapolate results to similar ungauged sites. Models may be used to extend time series of flows, stages, and quality parameters beyond the duration of measurements, from which statistical performance measures then may be derived. They may also be used for design optimization and real-time control.

This chapter presents modeling guidelines and describes some frequently-used operational (see Section IV for definition of "operational") models that are readily available for various kinds of analyses related to stormwater management. No attempt will be made to describe all available models, nor will details of individual models be presented.

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II. PROBLEM IDENTIFICATION

While models may be useful for the solution of many stormwater problems, the following types of problems have been found to be especially amenable to analysis using computer modeling.

A. Drainage and Flooding

The traditional urban drainage design problem of avoidance of street and surface flooding is commonly analyzed by models. Only a few models can analyze the general problem of surcharging, the condition in which the hydraulic grade line is above the sewer crown. Conditions during surcharging are hydraulically similar to pressurized conditions found in a water distribution network and often are modeled in a similar manner (see Chapter 6). Models can be used to simulate flows in both the major and minor systems, as well as the effect of possible inlet controls on combined sewers (Wisner 1982; Wisner et al. 1986).

B. Detention/Retention Storage

Many of the storm drainage models currently available can simulate the hydraulics of detention facilities. Simulation of the behavior of such facilities for removal of pollutants in urban runoff is in its infancy, due largely to the lack of scientifically controlled studies on the pollutant removal efficiency of these devices.

C. Sedimentation

Sedimentation in storm sewers can constitute both a quantity and quality problem. Excessive sedimentation can diminish the hydraulic capacity of the system and result in upstream flooding. Sediment deposited during small storms can flush during larger storm events. Simulation of the complex scour and deposition cycle of sediment found in sewers remains at a low stage of development. However, some empirical modeling and attempts at more deterministic modeling will be discussed later.

D. Water Quality

Studies and projects involving urban stormwater runoff quality can relate to many problems. In a narrower sense, a water quality study may address a particular issue, such as bacterial contamination of a beach, release of oxygen demanding material into a stream or river, unacceptable aesthetics of an open channel receiving urban runoff, eutrophication of a lake, contamination of basements from surcharged sewers due to wet-weather flooding, etc. In some instances, local or state regulations may prescribe a nominal "solution" without recourse to any water quality analysis as such (see Section IIIB).

Simulation of stormwater impacts on receiving water quality involves modeling of both quantity and quality. Superimposed on almost any water quality modeling effort is the need to analyze controls and abatement strategies. The considerable uncertainty in quality modeling makes the effort especially difficult.

III. URBAN MODELING OBJECTIVES AND CONSIDERATIONS

If a problem does require modeling, particular modeling objectives will probably result. Models may be used for objectives such as the following:

(a) Characterize the urban runoff as to temporal and spatial flow distributions, concentration/load ranges, etc.
(b) Provide input to a receiving water quality analysis, e.g., drive a receiving water quality model.
(c) Determine effects, magnitudes, locations, combinations, etc. of control options.
(d) Perform frequency analysis on hydrologic or quality parameters, e.g., to determine return periods of concentrations/loads.
(e) Provide input to economic analyses.

A. Planning, Analysis/Design, and Operation

Another way of looking at modeling objectives is to consider that there are several levels of the process—planning, analysis/design and operation—that can use modeling.

Planning involves a comparison of general design and/or flood mitigation strategies and may include optimization and risk assessment. At the planning level, the relative effectiveness of alternative drainage and flood control practices may be assessed and economic trade-offs evaluated. Modeling is likely to be somewhat less detailed in an effort to screen several alternative strategies. Continuous simulation (discussed below) can be useful at this level to determine relative flooding frequency as affected by alternative stormwater management programs, selection of hydrologic events for detailed design and assessment of the reliability of a proposed design, and economic optimization.

At the analysis/design level, the detailed analysis of an existing system, proposed system, or system improvements is investigated. Examples include analysis of alternative surface drainage patterns, location of detention storage facilities, and alternative runoff transport systems (e.g. swales vs. pipes). Design models must be capable of realistic simulation of hydrologic, hydraulic and, possibly, water quality phenomena.

Operational controls are devices that function during a storm such as variable weirs, pumps, and gates. These devices are most often found

in combined sewer systems and are used to minimize combined sewer overflows rather than provide flood protection. More information on combined sewer system analysis can be found in Chapter 10.

B. When Should A Model Be Used?

Modeling may not always be necessary to address the problems identified earlier or the objectives listed above. Since the process of implementing a model, collecting the necessary data, calibrating and verifying the model, and assessing the results can be very time consuming and expensive, it is important to understand that problems may be amenable to a simpler form of analysis. For example, field investigation of a flooding problem may reveal culverts choked by debris and/or sediment, or overgrown channels. Simple maintenance may solve this problem. In another case, field inspection may reveal that a single channel or road crossing is undersized. Upsizing this one structure to the capacity of the channel immediately upstream may solve the problem so that it is not necessary to model the entire upstream watershed. Modeling may also be unnecessary to implement required control measures.

By the same token, it should not be assumed that every water quality problem requires a water quality modeling effort. Some problems may be strictly hydraulic in nature, e.g., the basement flooding problem. That is, the solution may often reside primarily in a hydrologic or hydraulic analysis in which the concentration or load of pollutants is irrelevant.

For example, the State of Florida requires the capture of the first one-half inch of runoff for water quality control for certain size developments. Storage requirements for this case are simply calculated as one-half acre-inch per acre of development. In other cases the State requires that the runoff from the first inch of rainfall be captured for water quality control. For this case, a runoff model may be required to calculate the runoff volume produced by a one-inch rainfall.

While modeling generally yields more information, simpler methods may provide sufficient information for developing a control strategy. **In general, the simplest method that provides the desired analysis should be used.** The risk of using a more complex (and presumably "better") model is that it requires more expertise, data, support, etc. to use and understand, with a consequent higher probability of misapplication.

If modeling is necessary, it still may not be necessary to simulate quality processes since most control strategies are based on hydrologic or hydraulic considerations. Quality processes are very difficult to simulate accurately and generally incorporate many heuristic procedures that require extensive calibration (Huber 1985). If abatement strategies can be developed without the simulation of water quality parameters, the overall modeling program will be greatly simplified.

Computer models allow some types of analysis (such as frequency analysis) to be performed that could rarely be performed otherwise, since periods of runoff or quality measurements in urban areas are

seldom very long. It should always be borne in mind, however, that use of measured data is usually preferable to use of simulated data, particularly for objectives (a) and (b), in which accurate concentration values are needed. Modeling is **not** a good substitute for data collection, especially for water quality parameters. Although modeling is generally cheaper than data collection, the uncertainties involved, especially in water quality simulation, mandate the collection of data for model calibration and verification.

Models sometimes may be used to extrapolate beyond the measured data record. It is important to recognize, however, that models do not extend data, but rather generate mathematically simulated numbers that should never be assumed to be the same as data collected in the field.

Careful consideration should be given when using models to provide input to receiving water quality analyses. The first urban runoff quality model (SWMM) inadvertently overemphasized the concept of simulation of detailed intra-storm quality variations, e.g., production of a "pollutograph" (concentration vs. time) at 5 or 10 minute intervals during a storm for input to a receiving water quality model. The fact is, however, that the quality response of most receiving waters is insensitive to such short-term variations, as illustrated in Table 7.1. In most instances, the total storm load will suffice to determine the receiving water response, eliminating the need to calibrate against detailed pollutographs. Instead, only the total storm loads need be matched, a much easier task. Simulation of short time increment changes in concentrations and loads is generally necessary only for analysis of control options, such as storage or high-rate treatment, whose efficiency may depend on the transient behavior of the quality constituents.

Any consideration of water quality modeling means that some additional data will be required for model input. As described later, such

TABLE 7.1. Required Temporal Detail for Receiving Water Analysis (After Driscoll 1979; and Hydroscience 1979)

Type of Receiving Water (1)	Key Constituents (2)	Response Time (3)
Lakes, Bays	Nutrients	Weeks—Years
Estuaries	Nutrients, OD*	Days—Weeks
	Bacteria	
Large Rivers	OD, Nitrogen	Days
Streams	OD, Nitrogen	Hours—Days
	Bacteria	Hours
Ponds	OD, Nutrients	Hours—Weeks
Beaches	Bacteria	Hours

*OD = oxygen demand, e.g., BOD, that affects dissolved oxygen.

requirements may be as simple as a constant concentration or much more complex. Data may be obtained from existing studies or their acquisition may require extensive field monitoring. For some conceptualizations of the urban quality cycle, e.g., buildup and washoff, it may not be routinely possible to physically measure fundamental input parameters, and such parameters will only be obtained through model calibration. Involvement in acquisition of quality data, be it through literature reviews or field surveys, profoundly escalates the level of effort required for the study. Details on data requirements for urban areas will be deferred until modeling techniques are described.

IV. MODEL DEFINITION

In the broadest context, a model can be defined as any organized procedure for the analysis of a problem. With such a definition, almost any traditional technique could be included for discussion, from the rational method to unit hydrographs. However, this chapter treats a model in the popular sense of a computer program designed to analyze one or many problems encountered in storm drainage systems. The program may well incorporate traditional procedures, but will also include extensive routines for data management, including input and output procedures, and possibly including graphics and statistical capabilities. More specifically, this chapter discusses only those models that are "operational," i.e. defined as satisfying the following three criteria:

(a) An operational model must have documentation. This must include a user's manual that describes input data requirements, output to be expected, and computer requirements. In addition, the theory and numerical procedures used in the model must be explained so that the user will understand the basis for the model predictions. Documentation is the characteristic that most often distinguishes a model that can be accessed and used by others from the many computerized procedures described in the literature.

(b) An operational model must have support. This may be provided by the original model developer, the commercial vendor for the model or a sponsoring government agency. The outstanding hydrologic example is the Corps of Engineers, Hydrologic Engineering Center, in Davis, California which maintains a staff specifically for the purpose of model development, maintenance, and updating. Support means that the user can obtain answers, by telephone or written correspondence, to problems that arise during model implementation and use.

(c) An operational model should have been widely used by other than just the model developer. Regardless of its technical virtues, a procedure described in a single journal article or report with no experience or "review" by the engineering community is a weak candidate for use by a third party. Furthermore, user feedback is an invaluable means for identifying model limitations and "bugs," and initiating improvements and corrections to a model. Of course, no model will meet this

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third criterion initially, and the prospective user must decide on the relative merits of new options versus older ones. Several models satisfying these criteria will be discussed later.

V. OVERVIEW OF AVAILABLE MODELING OPTIONS

A. Introduction

Several classification schemes can be developed for models, to differentiate the type and versatility of various models, e.g., deterministic versus stochastic, transient versus steady-state, lumped versus distributed, number of dimensions, quality and/or simulation, etc. These classifications are relatively unimportant when considering simulation of drainage systems because nearly all models are transient and schematize the whole system by linking lumped or one-dimensional schematizations of individual components such as catchments or pipes. Thus, for drainage system simulation there are only a few major distinguishing features among models.

Quantity modeling is relatively well understood. Many models can convert rainfall into runoff and perform flow routing; the user can make a selection on the basis of method used, computer supported, options included, etc. A reasonably accurate prediction of a runoff hydrograph will result if the modeler knows just three input parameters: the catchment imperviousness (or directly connected or hydraulically effective imperviousness), the watershed area and the rainfall hyetograph. Given these three parameters, hydrograph volumes and peaks may be predicted within "ball park" accuracy even before calibration.

Quality modeling is quite different. In a review of quality modeling methodologies, Huber (1985) concluded that prediction of absolute values of concentrations of quality parameters was not possible without calibration and verification data. That is, first-cut modeling attempts may differ from "true" values by orders of magnitude for concentrations and loads. About all that might be safely concluded from quality modeling without measured calibration and verification data is the **relative** effect of control strategies, although even these might be open to question depending upon the assumptions incorporated into the model. The result is that simulation of quality parameters should be performed only when necessary, and only when requisite calibration and verification data are available.

Modeling of runoff water quality parameters or development of water quality input data are often a prelude to receiving water quality modeling. A discussion of receiving water quality models is beyond the scope of this chapter. However, reviews are provided by Hinson and Basta (1982), EPA (1983a), Barnwell (1984), Thomann and Mueller (1987) and Ambrose et al. (1988). Corps of Engineers models are summarized briefly in a brochure (U.S. Army Corps of Engineers 1987).

B. Continuous Versus Single-Event Simulation

The first major hydrologic model was the Stanford Watershed Model (Crawford and Linsley 1966). It simulated the long-term response (e.g., over several years) of a watershed to precipitation input. It was the first continuous model, and its hydrologic components may be found in the current HSPF model (Johanson et al. 1984). Subsequent models typically were single-event models that converted an individual rainfall hyetograph into an individual runoff hydrograph, perhaps with quality processes superimposed. Single-event models characteristically employ a short-time step (e.g. 5–15 min.) and a detailed catchment and drainage network schematization. These models are often used for detailed design. Continuous models use a longer time step (usually 1 hour to correspond to available recorded hourly rainfall data) and generally employ a coarse schematization of the catchment and drainage network to avoid excessive computer run times.

A distinct advantage of continuous simulation is its ability to provide antecedent conditions as an implicit component of the modeling. That is, continuous records of soil moisture, water table elevation, surface storage, surface pollutant loads, dry-weather activities, etc. may all be maintained, eliminating the vexing questions of initial conditions inherent in single-event modeling. Critical design conditions fall out naturally from a continuous analysis. If necessary, these conditions can later be simulated in more detail in a single-event model, since antecedent conditions can be obtained from the earlier continuous simulation. Advantages such as these are summarized by Linsley and Crawford (1974), and James and Robinson (1982), who discuss implications of continuous simulation on design of detention storage.

Continuous simulation is most useful for planning and optimization of the preliminary design; single-event simulation may then be subsequently used for detailed design and analysis. Individual models may perform both functions by varying the level of schematization and the time step. Output from continuous simulation may be analyzed as a time series and thus reveal more information than statistical methods (Goforth et al. 1983), although greater effort is generally required.

Single-event simulation has the advantage of well-defined data sets for calibration and verification and the ability to incorporate details of the sewer system that are usually too time-consuming or expensive to simulate in the continuous mode. In order to reap the benefits of continuous modeling at less cost and effort, combined approaches are sometimes used in which event simulation is performed on a number of storms and a frequency analysis performed on the subset of continuous meteorological input. Examples are provided by Walesh et al. (1979), Walesh and Snyder (1979) and Murphy et al. (1982).

C. Modeling Options

Several modeling options exist for simulation of quantity and quality in urban storm and combined sewer systems. These have been reviewed

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by Huber (1985, 1986) and range from simple to involved, although some "simple" methods, e.g., the EPA statistical methods, can incorporate quite sophisticated concepts. The principal methods available to the contemporary engineer are outlined generically below, in a rough order of complexity. Before choosing one of the methods, there are a number of selection criteria that must be considered. They include:

(a) Have a clear statement of project objectives. Verify the need for quality modeling. (Perhaps the objectives can be satisfied without quality modeling.)
(b) Use the simplest model that will satisfy the project objectives. Often a screening model, e.g., regression or statistical, can determine whether more complex simulation models are needed.
(c) To the extent possible, use a quality prediction method consistent with available data. This would often rule against buildup-washoff formulations, although these might still be useful for detailed simulation, especially if calibration data exist.
(d) Only predict the quality parameters of interest and only over a suitable time scale. That is, storm event loads and event mean concentrations (EMCs, the average concentration measured over the entire storm event) will usually represent the most detailed prediction requirement, and seasonal or annual loads will sometimes be all that are required. Do not attempt to simulate intra-storm variations in quality unless it is necessary.
(e) Perform a sensitivity analysis on the selected model and familiarize yourself with the model characteristics.
(f) If possible, calibrate and verify the model results. Use one set of data for calibration and another independent set for verification. If no such data exist for the application site, perhaps they exist for a similar catchment nearby.

The following methods are commonly used for runoff quality prediction. It should be noted that these methods all involve quantity and quality, and some are suitable for simulating quantity alone.

1. *Constant Concentration or Unit Loads*

As its name implies, constant concentration means that all runoff is assumed to have the same, constant concentration at all times for a given pollutant. At its very simplest, an annual runoff volume can be multiplied by a concentration to produce an annual runoff load. However, this option may be coupled with a hydrologic model, wherein loads (product of concentration and flow) will vary if the model produces variable flows. This option may be quite useful because it may be used with any hydrologic or hydraulic model to produce loads, merely by multiplying by the constant concentration. For instance, the highly sophisticated SWMM EXTRAN Block may be used for hydraulic analysis of a sewer system, prediction of overflows and diversions to receiving waters, etc., yet it performs no quality simulation as such. In many instances, it may be most important to get the volume and timing

of such overflows and diversions correctly, and simply estimate loads by multiplying by a concentration.

An obvious question is what (constant) concentration to use? Early (pre-1977) concentration and other data are summarized in publications such as Manning et al. (1977) and Lager et al. (1977). The more recent EPA NURP studies (U.S. EPA 1983) have produced a large and invaluable data base from which to select numbers, but the 30-city coverage of NURP will most often not include a site representative of the area under study. Nonetheless, a large data base does exist from which to review concentrations. Another option is to use measured values from the study area. This might be done from a limited sampling program. However, the NURP study conclusively demonstrated the variation that exists in EMCs at a site, within a city, and within a region or the country as a whole. Thus, while use of a constant concentration may produce **load** variations, EMC variations will not be replicated. These variations may be important in the study of control options and receiving water responses.

Unit loads are perhaps an even simpler concept. These consist of values of mass per area per time, typically lb/ac-yr or kg/ha-yr, for various pollutants, although other normalizations such as lb/curb-mile are sometimes encountered. Annual (or other time unit) loads are thus produced upon multiplication by the contributing area. Such loadings are obviously highly site-specific and depend upon both demographic and hydrologic factors. They must be based on an average or "typical" runoff volume and cannot vary from year to year, but they can conveniently be subject to reduction by best management practices (BMPs) if the BMP effect is known. Although early EPA references provide some information for various land uses (U.S. EPA 1973; U.S. EPA 1976a; McElroy et al. 1976), unit loading rates are exceedingly variable and difficult to transpose from one area to another. Constant concentrations can sometimes be used for this purpose, since mg/l x 0.2265 = lb/ac per inch of runoff. Thus, if a concentration estimate is available, the annual loading rate, for example, may be calculated by multiplying by the inches per year of runoff. Finally, the Universal Soil Loss Equation (Wischmeier and Smith 1958; Heaney et al. 1975)) was developed to estimate tons per acre per year of sediment loss from land surfaces. If a pollutant may be considered as a fraction ("potency factor") of suspended solids concentration or load, this offers another option for prediction of annual loads. Lager et al. (1977), Manning et al. (1977) and Zison (1980) provide summaries of such values.

2. *Spreadsheets*

Microcomputer spreadsheet software, e.g., Lotus, Quattro, Excel, is now ubiquitous in engineering practice. Very extensive and highly sophisticated engineering analysis is routinely implemented on spreadsheets, and water quality simulation is no exception. The spreadsheet most definitely may be used to automate and extend the concept of the

constant concentration or unit load idea. In the usual manifestation of this spreadsheet application, runoff volumes are calculated very simplistically, usually using a runoff coefficient times a rainfall depth. The coefficient may vary according to land use, or an SCS procedure may be used, but the hydrology is inherently simplistic in the spreadsheet predictions. The runoff volume is then multiplied by a constant concentration to predict runoff loads. Alternatively, unit loads are input directly and then multiplied by corresponding land use areas. The advantage of the spreadsheet is that a mixture of land uses (with varying concentrations or loads) may easily be simulated, and an overall load and flow-weighted concentration obtained from the study area (Walker et al. 1989). The study area itself may range from a single catchment to an entire urban area, and "delivery ratios" can be added to simulate loss of pollutants along drainage pathways between the simulated land use and the receiving waters. The relative contributions of different land uses may be easily identified, and handy spreadsheet graphics tools used for display of the results.

As an enhancement, control options may be simulated by application of a constant removal fraction for an assumed BMP. Although spreadsheet computations can be amazingly complex, BMP simulation is rarely more complicated than a simple removal fraction because anything further would require simulation of the dynamics of the removal device (e.g., a wet detention pond), which is usually beyond the scope of the hydrologic component of the spreadsheet model. Nonetheless, if simple BMP removal fractions can be believed, the spreadsheet can easily be used to estimate the effectiveness of control options. Loads with and without controls can be estimated and problem areas, by contributing basin and land use, can be determined. Since most engineers are familiar with spreadsheets, such models can be developed in-house in a logical manner.

The spreadsheet approach is best suited to estimation of long-term loads, such as annual or seasonal, because very simple prediction methods generally perform better over a long averaging time and poorly at the level of a single storm event. Hence, although the spreadsheet could be used at the microscale (at or within a storm event) it is most often applied for much longer time periods. It is harder to obtain the variation of predicted loads and concentrations using the spreadsheet method because this can ordinarily only be done by varying the input concentrations or rainfall values. A Monte Carlo simulation may be attempted (i.e., systematic variation of all input parameters according to an assumed frequency distribution) if the number of such parameters is not too large. These results may then be used to estimate the range and/or frequency distribution of predicted loads and concentrations.

In a generic sense, the spreadsheet idea may be used in methods programmed in other languages, e.g., Fortran. For example, comprehensive assessments of coastal zone pollution from urban areas are made by NOAA (1987) by assembling land use data with different runoff

coefficients, predicting daily and seasonal runoff volumes from daily rainfall, and predicting seasonal pollutant loads using constant concentrations. Although the demographic data base and use of magnetic tapes may dictate use of mainframes, the computational concept is still that of a spreadsheet.

Again, the question arises of what concentrations or unit loads to use, this time potentially for multiple land uses and subareas. And again, the NURP data base will usually be the first one to turn to, with the possibility of local monitoring to augment it.

3. *Statistical Approaches*

Uncertainties in deterministic or physically-based methods for water quality modeling have led to the use of statistical approaches as a data-based alternative to uncertain model predictions. These methods basically ignore the process conceptualizations employed by physically-based models and substitute generalizations about flow, pollutant concentration and load means, standard deviations, and probability distributions based on observations gathered at many sites.[1] The statistical approach leads to prediction of the probability distribution of event mean concentrations for the site in question. The key problem is how to determine the statistical properties of EMCs at ungauged sites, not a dissimilar problem from the use of physically-based models. Another problem with statistical approaches is studying the effect of control options or changes to the physical system. Nonetheless, the statistical approach has been used in several important applications.

Statistical analyses of various kinds are also incorporated into most evaluations of measured data. For instance, water quality data may be averaged over time intervals from the start of a storm event to identify first flush phenomena. Similarly, the concentration time series may reflect the storm hyetograph and hydrograph. Regression methods are often applied in order to extrapolate observed data to new areas (Driver and Tasker 1988, Tasker and Driver 1988), subject to the usual caveats about regression analysis.

The so-called "EPA Statistical Method" is somewhat generic and until recently was not implemented in any off-the-shelf model or even very well in any single report (Hydroscience 1979; U.S. EPA 1983). A new FHWA study (Driscoll et al. 1989) partially remedies this situation. The concept is straightforward, namely that of a derived frequency distribution for EMCs. This idea has been used extensively for urban runoff

[1]It should be noted that it is also possible to use a physically-based continuous simulation model to generate a time series of flows, pollutant concentrations, or loads, which can then be analyzed statistically.

quantity (e.g., Howard 1976; Loganathan and Delleur 1984; Zukovs et al. 1986) but not as much for quality predictions.

The EPA Statistical Method is based on the fact that EMCs are not constant but tend to exhibit a lognormal frequency distribution. When coupled with an assumed distribution of runoff volumes (also lognormal), the distribution of runoff loads may be derived. When coupled again to the distribution of streamflow, an approximate (lognormal) probability distribution of in-stream concentrations may be derived (Di Toro 1984)—a very useful result, although assumptions and limitations of the method have been pointed out by Novotny (1985) and Roesner and Dendrou (1985). Further analytical methods have been developed to account for storage and treatment (Di Toro and Small 1979; Small and Di Toro 1979). The method was used as the primary screening tool in the EPA NURP studies (U.S. EPA 1983) and has also been adapted to combined sewer overflows (Driscoll 1981) and highway-related runoff (Driscoll et al. 1989). This latter publication is one of the best for a concise explanation of the procedure and assumptions and includes spreadsheet software for easy implementation of the method.

A primary assumption is that EMCs are distributed lognormally at a site and across a selection of sites. The concentrations may thus be characterized by their median value and by their coefficient of variation (CV = standard deviation divided by the mean). There is little doubt that the lognormality assumption is good (Driscoll 1986), but similar to the spreadsheet approach, the method is then usually combined with weak hydrologic assumptions, e.g., prediction of runoff using a runoff coefficient (the accuracy of a runoff coefficient increases as urbanization and imperviousness increase.) However, since many streams of concern in an urban area consist primarily of stormwater runoff during wet weather, the ability to predict the distribution of EMCs is very useful for assessment of levels of exceedance of water quality standards. The effect of BMPs can again be estimated crudely through constant removal fractions that lower the EMC median, but it is harder to determine the effect on the coefficient of variation. Overall, the method has been very successfully applied as a screening tool.

Input to the method as implemented for the FHWA (Driscoll et al. 1989) includes statistical properties of rainfall (mean and CV of storm event depth, duration, intensity and interevent time), area, and runoff coefficient for the hydrologic component, plus EMC median and CV for the pollutant. Generalized rainfall statistics have already been calculated for many locations in the U.S. Otherwise, the EPA SYNOP model (U.S. EPA 1976b; Hydroscience 1979; U.S. EPA 1983; Woodward-Clyde 1989) must be run on long-term hourly rainfall records. If receiving water impacts are to be evaluated, the mean and CV of the streamflow are required plus the upstream concentration. A Vollenweider-type lake impact analysis is also provided based on phosphorus loadings.

As with the first two methods discussed, the choice of median concentration may be difficult, and the Statistical Method requires a coefficient of variation as well. Fortunately, from NURP and highway studies, CV values for most urban runoff pollutants are fairly consistent, and a value of 0.75 is typical. If local and/or NURP data are not available or inappropriate, local monitoring may be required, as in virtually every quality prediction method. The estimation of the whole EMC frequency distribution for a pollutant is a definite advantage of the Statistical Method over some applications of constant concentration and simple spreadsheet approaches. Frequency analyses of water quantity and quality parameters may also be performed on the output of continuous simulation models such as HSPF, SWMM, and STORM. The derived distribution approach of the Statistical Method avoids the considerable effort required for continuous simulation at the expense of simplifying assumptions that may or may not reflect the prototype situation adequately.

4. *Regression—Rating Curve Approaches*

With the completion of the NURP studies in 1983, there are measurements of rainfall, runoff and water quality at well over 100 sites in over 30 cities. Some regression analysis has been performed to try to relate loads and EMCs to catchment, demographic and hydrologic characteristics (e.g., McElroy et al. 1976; Miller et al. 1978; Brown 1984), the most recent of which is the work by the USGS (Tasker and Driver 1988; Driver and Tasker 1988), described briefly below. Regression approaches have also been used to estimate dry-weather pollutant deposition in combined sewers (Pisano and Queiroz 1977), a task at which no model is very successful. What are termed "rating curves" herein are just a special form of regression analysis, in which concentration and/or loads are related to flow rates and/or volumes. This is an obvious exercise attempted at most monitoring sites and has a historical basis in sediment discharge rating curves developed as a function of flow rate in natural river channels.

A rating curve approach is most often performed using total storm event load and runoff volume although intra-storm variations can sometimes be simulated in this manner as well (e.g., Huber and Dickinson 1988). It is usually observed (Huber 1980; U.S. EPA 1983; Driscoll et al. 1989) that concentration (EMC) is poorly or not correlated with runoff flow or volume, implying that a constant concentration assumption is adequate. Since the load is the product of concentration and flow, load is usually well correlated with flow regardless of whether or not concentration correlates well. This manifestation of spurious correlation (Bensen 1965) is often ignored in urban runoff studies. If load is proportional to flow to the first power (i.e. linear), then the constant concentration assumption holds; if not, some relationship of concentration with flow is implied. Rating curve results can be used by them-

selves for load and EMC estimates and can be incorporated into some models (e.g., SWMM, HSPF).

Rainfall, runoff and quality data were assembled for 98 urban stations in 30 cities (NURP and other) in the U.S. for multiple regression analysis by the USGS (Driver and Tasker 1988; Tasker and Driver 1988). Thirty-four multiple regression models (mostly log-linear) of storm runoff constituent loads and storm runoff volumes were developed, and 31 models of storm runoff EMCs were developed. Regional and seasonal effects were also considered. The two most significant explanatory variables were total storm rainfall and total contributing drainage area. Impervious area, land use, and mean annual climatic characteristics also were significant explanatory variables in some of the models. Models for estimating loads of dissolved solids, total nitrogen, and total ammonia plus organic nitrogen (TKN) generally were the most accurate, whereas models for suspended solids were the least accurate. The most accurate models were those for the more arid Western United States, and the least accurate models were those for areas that had large mean annual rainfall.

These USGS equations represent the best generalized regression equations currently available for urban runoff quality prediction. Note that such equations do not require preliminary estimates of EMCs or local quality monitoring data except for the very useful exercise of verification of the regression predictions. Regression equations only predict the mean and do not provide the frequency distribution of predicted variable, a disadvantage compared to the statistical approach (the USGS documentation describes procedures for calculation of statistical error bounds, however). Finally, regression approaches, including rating curves, are notoriously difficult to apply beyond the original data set from which the relationships were derived. That is, they are subject to very large potential errors when used to extrapolate to different conditions. Thus, the usual caveats about use of regression relationships continue to hold when applied to prediction of urban runoff quality.

5. *Buildup and Washoff*

In the late 1960s, a Chicago study by the American Public Works Association (1969) demonstrated the (assumed linear) buildup of "dust and dirt" and associated pollutants on urban street surfaces. During a similar time frame, Sartor and Boyd (1972) also demonstrated buildup mechanisms on the surface as well as an exponential washoff of pollutants during rainfall events. These concepts were incorporated into the original SWMM model, as well as into the STORM, USGS, and HSPF models to a greater or lesser degree (Huber 1985). "Buildup" is a term that represents all of the complex spectrum of dry-weather processes that occur between storms, including deposition, wind erosion, street cleaning, etc. The idea is simply that all such processes lead

to an accumulation of solids and other pollutants that are then "washed off" during storm events.

Although ostensibly physically based, models that include buildup and washoff mechanisms really employ conceptual algorithms because the true physics is related to principles of sediment transport and erosion that are poorly understood in this framework. Furthermore, the inherent heterogeneity of urban surfaces leads to use of average buildup and washoff parameters that may vary significantly from what may occur in an isolated street gutter, for example. Thus, except in rare instances of measurements of accumulations of surface solids, the use of buildup and washoff formulations inevitably results in a calibration exercise against measured end-of-pipe quality data. It then holds that in the absence of such data, inaccurate predictions can be expected.

Different models offer different options for conceptual buildup and washoff mechanisms, with SWMM having the greatest flexibility. In fact, with calibration, good agreement can be produced between predicted and measured concentrations and loads with such models, including intra-storm variations that cannot be duplicated with most of the methods discussed earlier. (When a rating curve is used in SWMM instead of buildup and washoff, it is also possible to simulate intra-storm variations in concentration and load.) A survey of linear buildup rates for many pollutants by Manning et al. (1977) is probably the best source of generalized buildup data, and some information is available in the literature to aid in selection of washoff coefficients (Huber 1985; Huber and Dickinson 1988). However, such first estimates may not even get the user in the ball park (i.e., quality—not quantity—predictions may be off by more than an order of magnitude); the only way to be sure is to use local monitoring data for calibration and verification. Thus, as for most of the other quality prediction options discussed herein, the buildup-washoff model may provide adequate comparisons of control measures, ranking of loads, etc., but cannot be used for prediction of absolute values of concentrations and loads, e.g., to drive a receiving water quality model, without adequate calibration and verification data. Since buildup and washoff are somewhat appealing conceptually, it is somewhat easier to simulate potential control measures such as street cleaning and surface infiltration using these mechanisms than with, say, a constant concentration or rating curve method. In the relatively unusual instance in which intra-storm variations in concentration and load must be simulated, as opposed to total storm event EMC or load, buildup and washoff also offer the most flexibility. This is sometimes important for the design of storage facilities in which first-flush mechanisms may be influential.

As mentioned above, generalized data for buildup and washoff are sparse (Manning et al. 1977) and such measurements are almost never conducted as part of a routine monitoring program. For buildup, normalized loadings, e.g., mass/day-area or mass/day per curb-length, or just mass/day, are required, along with an assumed functional form for buildup vs. time, e.g., linear, exponential, Michaelis-Menton, etc. For

washoff, the relationship of washoff (mass/time) vs. runoff rate must be assumed, usually in the form of a power equation. When end-of-pipe concentration and load data are all that are available, all buildup and washoff coefficients end up being calibration parameters.

6. *Related Mechanisms*

In the discussion above, washoff is assumed proportional to the runoff rate, as for sediment transport. Erosion from pervious areas may instead be proportional to the rainfall rate. HSPF does the best job of including this mechanism in its algorithms for erosion of sediment from pervious areas. SWMM includes a weaker algorithm based on the Universal Soil Loss Equation (Wischmeier and Smith 1958; Heaney et al. 1975).

Many pollutants, particularly metals and organics, are adsorbed onto solid particles and are transported in particulate form. The ability of a model to include "potency factors" (HSPF) or "pollutant fractions" (SWMM) enhances the ability to estimate the concentration or load of one constituent as a fraction of that of another, e.g., solids (Zison 1980).

The groundwater contribution to flow in urban areas can be important in areas with unlined and open channel drainage. Of the urban models discussed, HSPF far and away has the most complex mechanisms for simulation of subsurface water quality processes in both the saturated and unsaturated zones (there are rarely, if ever, adequate local data to calibrate all HSPF parameters, and the user is forced to accept default values). Although SWMM includes subsurface flow routing, the quality of subsurface water can only be approximated at present using a constant concentration.

The precipitation load may be input in some models (SWMM, HSPF), usually as a constant concentration. Point source and dry-weather flow (baseflow) loads and concentrations can also be input to SWMM, STORM and HSPF to simulate background conditions. Other quality sources of potential importance include catchbasins (SWMM) and snowmelt (SWMM, STORM, HSPF).

Scour and deposition within the sewer system can be very important in combined sewer systems and some separate storm sewer systems. The state of the art in simulation of such processes is poor (Huber 1985), though SWMM offers a crude but calibratable attempt.

In the last analysis, the most complicated model may not be the most accurate one when estimating runoff quantity and quality. It is the experience and understanding of the engineer that is most important when generating runoff quantity and quality calculations.

VI. COMPUTER REQUIREMENTS

Virtually every engineer has access to a microcomputer, and most major hydrologic models have been modified to run on these systems. This model availability, coupled with advances in data acquisition and

management and the extraordinary graphical capabilities of microcomputers, means that the most advanced technology and largest data bases are now accessible to almost anyone. However, there are still advantages and disadvantages of both mainframe and microcomputing. Micros are "personal" and permit immediate access and "turnaround" (the time taken to run a program). Use of micros is inherently interactive, which encourages inspection and evaluation of data and results. Other microcomputer software, especially spreadsheets, makes it easy to perform simple calculations without the need of more complex models at all and without the need for programming (although spreadsheet manipulation is essentially the same thing as programming). Disadvantages of micros include execution time for long programs, and lengthy waiting periods for input and output of large data sets.

Mainframes have the advantages of speed, including fast peripherals such as high-speed printers and tape drives. Thus, a program that may require several hours to execute on a micro may execute in seconds on a mainframe.

Micros continue to increase in their performance, and already will satisfy most engineering needs. Most of the models to be discussed in this chapter are available in both mainframe and microcomputer versions.

VII. STEPS IN MODELING

A. Data Requirements

Before a model is even considered, some data will be available to indicate that there is a problem. For instance, receiving water samples may indicate a bacteriological problem, inspection may reveal sedimentation, or high-water marks may be seen on basement walls. Receiving water evaluations may include common parameters such as DO, oxygen demand, nutrients, and bacteria as well as a bio-assay or other biological assessment of receiving water quality. Such observations constitute a data set in and of themselves, and usually indicate the direction for subsequent data collection. At every stage of the preliminary analysis, one must ask if measured data can resolve the problem. If so, there is no need to model.

If modeling is required, there are three types of required data; model input data, calibration data, and verification data. Input data consist simply of the required parameters to run the model, and typically include rainfall information, area, imperviousness, runoff coefficient and other quantity prediction parameters, plus quality prediction parameters such as constant concentration, constituent median and CV, regression relationships, buildup and washoff parameters, etc.. Calibration is the process of parameter adjustment to obtain a match between predicted and measured output. Verification holds the parameters constant and tests the calibration on an independent data set.

Thus, calibration and verification data consist of the very elements the model predicts, such as hydrographs or pollutographs (a plot of concentration versus time), whereas input data may resemble physical characteristics of the system. Some input data for some models are mostly conceptual, such as a time of concentration, and cannot be readily measured. Calibration is used to estimate the value of these parameters, and verification is used to test the validity of the estimate.

Data sets which can be used for calibration and verification exist (e.g., Huber et al. 1982; Driver et al. 1985; Noel et al. 1987) but seldom for the site of interest. If the project objectives absolutely require such data (e.g., if a model must be calibrated to drive a receiving water quality model), then expensive local monitoring may be necessary.

B. Basic Input Data

All models require some form of input data. For quantity simulation, these data include catchment areas, imperviousness, slopes, roughnesses, etc.; channel and conduit linkages, shapes, sizes, slopes, roughnesses; invert and ground elevations; characteristics of hydraulic structures or controls such as weirs, orifices and pumps; depth-area-volume-outflow relationships for storage units; information on downstream hydraulic controls, such as river stages or tidal elevations. Since the overall system is driven by rainfall, suitable rainfall hyetographs must be found (see below), as well as base flow, if any, in the drainage channels.

Not all models need all these data; some use very simple methods to convert rainfall into runoff, but greater model complexity generally means greater model input requirements. A critical factor in successful hydraulic modeling of older drainage systems is an accurate survey to determine invert elevations and conduit or channel condition. These are seldom the same as shown on as-built plans because of settlement, deterioration, and modifications to the system. This is typically an expensive component of data preparation.

Dry-weather flow concentrations must be measured to simulate the mix of stormwater and base flow that occurs during a storm, as well as characteristics of solids if scour and deposition are to be simulated.

C. Rainfall Input

Rainfall is the driving force for all hydrologic simulation models. If adequate measured rainfall is not available, a good calibration between measured and predicted hydrographs cannot be expected. Even though the Stanford Watershed Model introduced the concept of continuous simulation to hydrologic modeling, most models have evolved from single-event formulations, for which a single hyetograph is used to simulate runoff. For calibration purposes, measured rainfall must be input to produce output for comparison with measured hydrographs. However for design purposes, synthetic design storms have mostly

replaced historic rainfall hyetographs as input to models (Keifer and Chu 1957; USDA 1971; Arnell 1982) because of the ease with which they may be constructed. The perils of single design storms are many, as discussed by McPherson (1978), not the least of which is the necessity for choosing antecedent conditions and the assumption that the return period of the rainfall input will be the same as the return period of the runoff (or pollutant) output.

Continuous simulation or statistical methods offer alternatives for selection of design rainfall. For example, a selection of historic storms can be made from a continuous simulation on the basis of the return period of the runoff or quality parameter of interest, e.g., peak flow, maximum runoff volume, maximum stage, peak runoff load, peak runoff concentration (Huber et al. 1986b). These events, with their antecedent conditions for runoff and quality, can then be analyzed in more detail in a single-event mode. Rainfall is variable in space as well as in time; models that accept multiple hyetographs can simulate storm motion and spatial variation that can strongly affect runoff hydrographs (Surkan 1974; James and Shtifter 1981). Many considerations related to the selection of rainfall input for modeling are provided in proceedings edited by Harremöes (1983).

D. Sensitivity Analysis

Before attempting to calibrate and verify a model, the user should be familiar with its capabilities and nuances. Some models have very few parameters to adjust, simplifying the calibration process, but others may have 30 or more. Which ones affect the output the most? Since this is seldom documented adequately, the user should perform a sensitivity analysis (with hypothetical data if necessary), varying key parameters by known percentages and inspecting the change in output. In this way, it will be far easier to know which parameters should be changed during the calibration process. For instance, in urban areas, most models are highly sensitive to imperviousness but only weakly sensitive to soil infiltration parameters. First runs with any model should deal with a very simple configuration for which the result is known, e.g., steady rain on an impervious surface, and build up gradually to more complex and realistic systems. In this way, the user can exercise good judgment regarding the validity and reasonableness of the results.

E. Calibration

Model calibration consists of adjusting model parameters (e.g., imperviousness, roughness) until the predicted output agrees with measured observations. For example, the predicted hydrograph or pollutograph may be adjusted to agree with the measured hydrograph or pollutograph. For most models, calibration will be performed using observed storm events. How many storms are required cannot be answered exactly, but 3–6 are desirable. The calibration process should

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be performed simultaneously for all available storms in order to produce a robust calibration (Maalel and Huber 1984). In this instance, the single set of calibration parameters will result in less-than-perfect fits for any single storm but better for all storms together, and presumably better for further predictions.

Calibration tends to be subjective. For example, how well does one hydrograph match another? When several storms are used, it is customary to plot predicted versus measured peaks and predicted versus measured volumes, seeking to produce points that fall on the 45-degree line indicating perfect agreement. Deviations from the line of perfect fit are one measure of the goodness of fit; hydrographs can (and should) also be compared visually. James and Burges (1982) furnish many practical guidelines for calibration, testing, and quantification of errors. Thomann (1982) provides guidelines for assessing goodness of fit, and Reckhow et al. (1990) have expanded upon Thomann's work.

During the calibration process, care must be taken to make sure that the physical parameters are not adjusted outside their reasonable range to achieve a "calibration." For example, if the Manning roughness coefficient for a concrete pipe has to be set at 0.10 to achieve calibration, most likely there is an error in the input data of some other variable such as pipe slope or model conceptualization.

Calibration usually provides the only means for determining values for input parameters related to water quality, such as buildup rates and washoff coefficients. Although limited measurements of surface constituents have been conducted (see Manning et al. 1977 and Terstriep et al. 1982), such data are generally useful only for a first parameter estimate. Quality concentrations and loads are so difficult to predict that calibration data provide almost the only means for parameter estimation (Huber 1985).

F. Verification

Ideally, an equal number of storms should be used for verification as for calibration; however, 1–3 often seems to be the pragmatic limit of the number of storms that can be afforded for this purpose. Goodness of fit may be assessed similarly to the method used for calibration. In the not unlikely event that the verification is poor, an improved calibration can be attempted. This is sometimes performed using a different grouping of storms for calibration and verification than was used during the first attempt.

G. Uncertainty Analysis

Uncertainty analysis is rapidly moving from state-of-the-art to state-of-practice. Uncertainty analysis can be used to compute expected output variability as a function of ill-defined input parameters. Additionally, this technique can serve as a means of quantifying model

acceptability (does the range of probable model outcomes intersect the range of observations).

Uncertainty analysis has been performed via first-order approximations and Monte Carlo simulation techniques. First-order uncertainty analysis provides estimates of the mean and standard deviation for the dependent variable (model output) as functions of the mean and standard deviation associated with each independent variable. While this technique is relatively fast computationally, it does require evaluation of all partial derivatives for the dependent variable with respect to each independent variable. Additionally, the assumptions inherent in first-order uncertainty analysis (system linearity, and type of output distribution) can cause erroneous estimations of true model output behavior.

Monte Carlo analysis is relatively simple to program, and does not suffer from the aforementioned assumptions. Unfortunately, this technique can be rather computer-intensive. Monte Carlo analysis involves the random selection of input parameter values from prescribed statistical distributions followed by model execution. The process is repeated until an adequate description of model output has been obtained. The required number of Monte Carlo iterations is a function of the questions being asked (mean response, 95% confidence intervals, complete model output probability density function) and the level of accuracy needed. Simply stated, more iterations will result in a better definition of model output variability. The user must, through prior experimentation, ascertain the appropriate number of iterations required.

Uncertainty analysis can be particularly useful in evaluating the relationship between field sampling and modeling. Hypothetical sampling scenarios can be tested to understand the expected uncertainty in model output. If the level of output variability is too large, the sampling strategy can be increased until an acceptable level of model output uncertainty is achieved. Finally, uncertainty analysis can also be used to quantify model acceptability (expansion of goodness-of-fit testing). Warwick and Wilson (1990) used uncertainty analysis, applied to the STORM model, to determine if the use of default areal pollution accumulation rates were acceptable for the site under investigation. The use of these default values was rejected because the range of probable STORM output did not encompass field observations of pollutant runoff concentrations. The use of areal accumulation rates taken from a nearby location could not be rejected based upon this test, and was thereby viewed as acceptable for future predictions.

H. Production Runs

Following the calibration and verification processes, the model is ready for application to the problem. During this phase, just as earlier, all model parameters and results should be double-checked for reasonableness. Continuity checks built into a model often aid in checking

results so that an unrealistic gain or loss of water (or pollutants) can be noticed. Violations of continuity sometimes indicate numerical problems, especially with sophisticated hydraulic routing models.

VIII. MODELS

A. Published Reviews

Several publications provide reviews of available models. Recent reviews that consider surface runoff quantity/quality models include Huber and Heaney (1982), Kibler (1982), Whipple et al. (1983), Barnwell (1984, 1987), Huber (1985, 1986), Bedient and Huber (1988), Viessman et al. (1989), WPCF (1989), and Donigian and Huber (1990). Renard et al. (1982) review water resources models, some of which are useful in urban hydrology. HEC models are described in detail by Feldman (1981). Descriptions of EPA nonpoint source water quality models are provided by Ambrose et al. (1988) and Ambrose and Barnwell (1989).

B. Models To Be Reviewed

There are so many potential models that might be used for analysis of storm drainage systems that some screening is in order. For this chapter, the most significant criterion is that the model be operational, as defined earlier, and have a fairly large user base. That still leaves potentially many models, at least if rainfall-runoff and other quantity-only models are to be included. Modeling and software development is highly dynamic, and the following model descriptions may be quite different in the future.

Nine of the models reviewed below are listed in Table 7.2, which characterizes the models in terms of various criteria. The other model, HEC-2, is reviewed because of its importance in simulating hydraulic processes, though it, like HEC-1, has no quality modeling capabilities.

1. DR3M-QUAL

U.S. Geological Survey has updated earlier model development by Dawdy et al. (1972) into the Distributed Routing Rainfall Runoff Model (DR3M), including quality, designed specifically for urban hydrology (Alley and Smith 1982a, 1982b). Runoff generation and subsequent routing use the kinematic wave method, and parameter estimation assistance is included in the model. Quality is simulated using buildup and washoff functions, with settling of solids in storage units dependent upon a particle size distribution. The model has been used in some of the EPA Nationwide Urban Runoff Program (NURP) studies that were conducted by the USGS (U.S. EPA 1983; Alley 1986). The FORTRAN version of the model is available from the U. S. Geological Survey's

TABLE 7.2. Comparison of Model Attributes.

Attribute (1)	Model: DR3M-QUAL (2)	HSPF (3)	ILLUDAS (4)	Penn State (5)	Statistical (6)	STORM (7)	SWMM (8)	TR55 (9)	HEC-1 (10)
Sponsoring agency	USGS	EPA	Ill. State Water Survey	OWRT and City of Phil.[a]	EPA	HEC	EPA	SCS	HEC
Simulation type[b]	C,SE	C,SE	SE	SE	N/A	C	C,SE	SE	SE
No. pollutants	4	10	None[c]	None	Any	6	10	None	None
Rainfall/runoff analysis	Y	Y	Y	Y	N	Y	Y	Y	Y
Sewer system flow routing	Y	Y	Y	N	N/A	N	Y	Y	Y
Full, dynamic flow routing equations	N	N	N	N	N/A	N	Y[d]	N	N
Surcharge	Y[e]	N	Y[e]	N	N/A	N	Y[d]	N	N
Regulators, overflow structures, e.g., weirs, orifices, etc.	N	N	N	N	N/A	Y	Y	N	N
Special solids routines	Y	Y	N/A	N/A	N	N	Y	N/A	N/A

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Storage analysis	Y	Y	Y	Y	Y[f]	Y	Y	Y	Y
Treatment analysis	Y	Y	N/A	N/A	Y[f]	Y	Y	N/A	N/A
Suitable for planning (P), design (D)[g]	P,D	P,D	D	D	P	P	P,D	D	D
Available on microcomputer	N	Y	Y	Y	Y[h]	N	Y	Y	Y
Data and personnel requirements	Medium	High	Low	Low	Medium	Low	High	Medium	Medium
Overall model complexity[i]	Medium	High	Low	Low	Medium	Medium	High	Low	High

[a]Currently supported by Penn State University.
[b]Y = yes, N = no, N/A = not applicable, C = continuous simulation, SE = single event simulation.
[c]Undocumented quality routines added during applications.
[d]Full dynamic equations and surcharge calculations only in Extran Block of SWMM.
[e]Surcharge simulated by storing excess inflow at upstream end of pipe. Pressure flow not simulated.
[f]Storage and treatment analyzed analytically. See references in Section 3.7.9.
[g]See Section 3.3.
[h]See Driscoll et al. 1989.
[i]General requirements for model installation, familiarization, data requirements, etc. To be interpreted only very generally.
[j]Reflection of general size and overall model capabilities. Note that complex models may still be used to simulate very simple systems with attendant minimal data requirements.

National Center in Reston, Virginia. No microcomputer version is available.

2. *HSPF*

The Hydrologic Simulation Program—Fortran (HSPF) was developed from hydrologic routines that began with the Stanford Watershed Model (Crawford and Linsley 1966) and nonpoint source water quality routines that were included in such models as the EPA Nonpoint Source Model (NPS), (Donigian and Crawford 1976) and the Agricultural Runoff Model (ARM) (Donigian and Davis 1978). The user's manual (Johanson et al. 1984) includes information on all hydrologic and water quality routines, including the "IMPLND" (impervious land) segment for use in urban areas. Additional guidelines for application are provided by Donigian et al. (1984). The model has special provisions for management of time series that result from the continuous simulation.

HSPF includes subsurface water balance and quality routing and contains the most comprehensive pollutant kinetics of any of the models discussed. The model is maintained by, and mainframe and microcomputer versions are available from, the U. S. Environmental Protection Agency, Athens, Georgia.

3. *ILLUDAS*

The Illinois Urban Drainage Area Simulator (ILLUDAS) (Terstriep and Stall 1974) evolved from the British Road Research Laboratory Model (Watkins 1962; Stall and Terstriep 1972). The model uses time-area methods to generate hydrographs from the directly connected paved area and from the pervious area. For pervious areas, the Horton infiltration equation is used to generate typical infiltration rates based on input of the soil's SCS hydrologic group category. A design routine is included that will re-size pipes of insufficient hydraulic capacity. User-provided stage/discharge/storage relationships are used to provide detention facilities anywhere in the system. Plots of calculated and observed hydrographs may be produced. Its simplicity and metric option have given ILLUDAS widespread use. Although quality is not formally included in the model, it has been added for special applications (Noel and Terstriep 1982).

4. *Penn State*

The Penn State Urban Runoff Model (PSURM) (Aron 1987) was originally developed in cooperation with the City of Philadelphia for drainage analysis, and has been applied to combined sewers in that city. Nonlinear reservoir routing is used for generation of the runoff hydrograph, coupled with a user-defined lag for routing within the sewer system. A hydraulic design capability helps to size pipes. No quality routines are included. The model has seen considerable use in the

northeastern United States (Kibler and Aron 1980, and Kibler et al. 1981). The PSURM is available in mainframe and microcomputer versions.

5. *Statistical*

The statistical approach is not a simulation model in the conventional sense, but rather a combined sequence of analysis of rainfall, runoff, and quality data coupled with analytical solutions (see extended discussion in Section V.C.3. of this chapter). The best explanation and implementation of the statistical method is in an adaptation developed for the U. S. Federal Highway Administration (Driscoll et al. 1989). This publication is one of the best for a concise explanation of the procedure and assumptions and includes spreadsheet software for easy implementation of the method. A useful component is the SYNOP ("synoptic precipitation") model for statistical analysis of hourly rainfall time series from magnetic tapes and floppy disks supplied by the National Climatic Data Center of NOAA at Asheville, NC.

6. *STORM*

The first significant use of continuous simulation in urban hydrology came with the Storage, Treatment, Overflow, Runoff Model (STORM), a program whose development was funded by the Corps of Engineers, Hydrologic Engineering Center (HEC 1977; Roesner et al. 1974). Early applications included the San Francisco master plan for combined sewer overflow (CSO) pollution abatement (McPherson 1974). The support of the HEC led to the wide use of STORM for planning purposes, especially for evaluation of the trade-off between treatment and storage as CSO control options (e.g., Heaney et al. 1977). Statistics of long-term runoff and quality time series permit optimization of control measures.

Although designed originally for analysis of CSOs, the model is equally useful for evaluating the effectiveness of detention facilities in reducing the frequency of runoff peaks for longer catchments.

STORM uses a simple runoff coefficient method for generation of hourly runoff depths from hourly rainfall inputs, and uses the buildup and washoff formulation for simulation of six pre-specified pollutants. However, the model can be manipulated to provide loads for arbitrary pollutants (Najarian et al. 1986). A microcomputer version has been demonstrated (Bontje et al. 1984), and a mainframe version is available. The HEC has also provided application guidelines (Abbott 1977).

7. *SWMM*

The original version of the Storm Water Management Model (SWMM) was developed for EPA as single-event model specifically for the analysis of combined sewer overflows (Metcalf and Eddy Inc. 1971). Through continuous maintenance and support, the model now is well suited to all types of storm water management from urban drainage to flood

routing and floodplain analysis. Version 4 (Huber and Dickinson 1988; Roesner et al. 1988) performs both continuous and single-event simulation throughout the whole model; can simulate backwater, surcharging, pressure flow and looped connections (by solving the complete dynamic wave equations in its Extran Block); and has a variety of options for quality simulation, including traditional buildup and wash-off formulations as well as rating curves and regression techniques. Subsurface flow routing (constant quality) may be performed in the Runoff Block in addition to surface quantity and quality routing, and treatment devices may be simulated in the Storage/Treatment Block using removal functions and sedimentation theory. A hydraulic design routine is included for sizing of pipes, and a variety of regulator devices may be simulated, including orifices (fixed and variable), weirs, pumps, and storage. A bibliography of SWMM usage is available (Huber et al. 1986) that contains many references to case studies.

SWMM is segmented into the Runoff, Transport, Extran, Storage/Treatment and Statistics blocks for rainfall-runoff, routing, and statistical computations. Water quality may be simulated in all blocks except Extran, and metric units are optional. Since the model is non-proprietary, portions have been adapted for various specific purposes and locales by individual consultants and other federal agencies, e.g., FHWA. Mainframe and microcomputer versions are available from EPA in Athens, Georgia.

8. TR55

The original Soil Conservation Service (SCS) methodology developed for general application (USDA 1971) was later adapted specifically to urban areas, and the latter procedure has come to be known as TR55 (USDA 1975). An updated user's manual is available (USDA 1986a), along with a microcomputer version (USDA, 1986b). (The 1975 version was not computerized by the SCS, although microcomputer versions are available from private vendors.)

Unit hydrographs are used to convert rainfall into runoff. If required, flow routing in channels must be performed separately by another model such as by the companion TR20 program (USDA 1983). SCS methods are widely used in the United States due to the wealth of soil information provided by the agency. Additional background on the method is provided by Viessman et al. (1989) and McCuen (1982). Information on application is usually available from local SCS offices as well.

9. HEC-1

The HEC-1 model developed by the Corps of Engineers Hydrologic Engineering Center (HEC 1985) is designed to simulate the surface runoff response of a river basin to precipitation by representing the basin as an interconnected system of hydrologic and hydraulic com-

ponents. Each component models an aspect of the precipitation-runoff process within a portion of the basin, commonly referred to as a subbasin. A component may represent a surface runoff entity, a stream channel, or a reservoir. The result of the modeling process is the computation of streamflow hydrographs at desired locations in the river basin. Multiplan-multiflood analysis allows the simulation of up to nine ratios of a design flood for up to five different plans (or characterizations) of a stream network in a single computer run. Dam-break simulation provides the capability to analyze the consequences of dam overtopping and structural failures. The depth-area option computes flood hydrographs preserving a user-supplied precipitation depth versus area relation throughout a stream network.

10. HEC-2

This program developed by the Corps of Engineers Hydrologic Engineering Center (HEC 1982), is intended for calculating water surface profiles for steady, gradually varied flow in natural or man-made channels. Both subcritical and supercritical flow profiles can be calculated. The effects of various obstructions such as bridges, culverts, weirs, and structures in the flood plain may be considered in the computations. The computational procedure is based on the solution of the one-dimensional energy equation with energy loss due to friction evaluated with Manning's equation. The computational procedure is generally known as the standard-step method (see Chapter 6). The program is also designed for application in flood plain management and flood insurance studies to evaluate floodway encroachments and to designate flood hazard zones. It is also capable of assessing the effects of channel improvements and levees on water surface profiles. Input and output units may be either U.S. customary units or metric.

Two supplementary programs are also available:

(a) A data edit program which checks the data cards for various input errors.
(b) A Fortran graphics program which produces HEC-2 cross section and profile plots in interactive or batch modes.

HEC-1 and HEC-2 are supported by the HEC and available from several private vendors.

IX. MODEL SELECTION

The brief abstracts in the previous section, information in Table 7.2, and the comparative reviews referenced previously may help in selection of a model, but the choice is often made on much more pragmatic grounds. For instance, an agency may specify that a certain model be used, or local support may be available. Probably the most important

ments in both software and computer hardware mean that, in the future, the possibilities will be limited only by the imagination and skill of the user.

Geographic information systems (GIS) enable the user to incorporate a wide range of information about the physical system into a computer data base. This can include not only information about the ground surface, but details of the urban infrastructure (water/wastewater, streets, electric, gas). The potential exists to integrate all the elements described above into a complete mapping and hydrology/hydraulics analysis and design package that can:

(a) Provide watershed physical feature mapping.
(b) Compute hydrologic model input parameters from the GIS.
(c) Model the rainfall/runoff process to determine design flows.
(d) Provide the capability for on-screen design of the system, including conveyance structures and appurtenances.
(e) Optimize the final design.
(f) Map or draw the system as designed, including plan and profile drawings of all structural components.

It should be noted that the requirements for checking and verification of designs so developed will still be necessary (or perhaps even more important).

XI. SUMMARY

Some form of modeling will almost assuredly become part of routine analyses performed at some portion of the thousands upon thousands of CSO and stormwater discharge locations around the country. Several modeling options exist, but none of them are truly "deterministic" in the sense of fully characterizing the physical, chemical, and biological mechanisms that underlie conceptual buildup, erosion, transport, and degradation processes that occur in an urban drainage system. Even if fully deterministic models were available, it is doubtful that they could be routinely applied without calibration data. But this is essentially true of almost all methods. Because a method is simple, e.g., constant concentration, does not make it more correct. Rather, the assumption is made that there will be some error in prediction regardless of the method, and there may be no point in compiling many hypothetical input parameters for a more complex model lacking a guarantee of a better prediction. For example, a study in Denver showed that regression equations could predict about as well as DR3M-QUAL given the available quality information (Ellis and Lindner-Lunsford 1986). But physically-based (conceptual) models do have certain advantages, discussed below.

Physically-based models depend upon conceptual buildup and washoff processes incorporated into the quality algorithms. Such models

have withstood the test of time and have been applied in major urban runoff quality studies. However, the relative lack of fundamental data on buildup and washoff parameters has led to simpler methods more often being applied, starting with the assumption of a constant concentration and becoming more complex. For example, the derived distribution approach of the EPA Statistical Method provides very useful screening information with minimal data; but it requires more data than by just assuming a constant concentration. With the mass of NURP and other data, regression approaches are now more viable but still subject to the usual restrictions of regression analysis. Spreadsheets are ubiquitous on microcomputers and serve as a convenient mechanism to implement several of the simple approaches, especially those that rely upon sets of coefficients, unit loads and/or EMCs as a function of land use or other demographic information.

Minimal data requirements and ease of application are the principal advantages of simpler simulation methods (constant concentration, unit loads, statistical, regression). However, in spite of their more complex data requirements, conceptual models (DR3M-QUAL, HSPF, STORM, SWMM) have advantages in terms of simulation of routing effects and control options as well as the superior statistical properties of continuous time series. For example, the EPA Statistical Method assumes that stream flow is not correlated with the urban runoff flow. This may or may not be true in a given situation, but it is not necessary to require such an assumption when running a model such as HSPF or SWMM. The four conceptual models discussed in detail all have a means of simulating storage and treatment effects. Other than a constant removal, this is difficult to do with the simpler methods. The conceptual quality models generally have very much superior hydrologic and hydraulic simulation capabilities (not true for STORM except that it can also use real rainfall hyetographs as input). This alone usually leads to better prediction of loads (product of flow times concentration). It should also be borne in mind that even complex models such as SWMM and HSPF can be run with minimal quality (and quantity) data requirements, such as using only a constant concentration. Finally, some of the case studies imply that transferability of coefficients and parameters is easier with buildup and washoff than with rating curve and constant concentration methods.

If a more complex conceptual model is to be applied, which one should it be from among those described herein? SWMM is certainly the most widely used and probably the most versatile for urban areas. HSPF may be more appropriate in areas with more open space where groundwater contributions increase in importance, where rainfall-induced erosion occurs, or where quality interactions are important along the runoff pathway. The simplicity of STORM remains attractive, and various consultants have used their own version as a planning tool. The USGS DR3M-QUAL model has been successfully applied in several USGS studies but has not seen much use outside the agency.

It contains useful techniques for quality calibration. SWMM and HSPF get limited support from the EPA Center for Exposure Assessment Modeling (CEAM) at Athens, Georgia. STORM and DR3M-QUAL will remain useful, but it is unlikely that either of these two models will enjoy enhancements or support from their sponsoring agencies in the near future.

What is a reasonable approach to simulation of urban runoff quality? The main idea is to use the simplest approach that will address the project objectives at the time. This usually means to start simple with a screening tool such as constant concentration (usually implemented in a spreadsheet) or regression or statistical approach. If these methods indicate that more detailed study is necessary or if they are unable to address all the aspects of the problem, e.g., the effectiveness of control options, then one of the more complex models must be run. No method currently available (or likely to be available) can predict absolute (accurate) values of concentrations and loads without local calibration data, including complex buildup and washoff models. Thus, if a study objective is to provide input loads to a receiving water quality model, local site-specific data will probably be required. On the other hand, several methods and models might be able to compare the relative contributions from different source areas, say, or to determine the relative effectiveness of control options (if the controls can be characterized by simple removal fractions). When used for purposes such as these, the methods, including buildup and washoff models, can usually be initiated on the basis of NURP and/or the best currently available source of quality data.

When properly applied and their assumptions respected, models can be tremendously useful tools in analysis of urban runoff quality problems. Methods and models are evolving that use the large current data base of quality information. As increasing attention is paid to urban runoff problems in the future, the methods and models can only be expected to improve.

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Chapter 8

DESIGN OF DRAINAGE CONVEYANCES

I. INTRODUCTION

A typical urban drainage system has two separate and distinct components, the major and the minor systems. Storm sewers, a part of the minor system, often have been the only **planned** portion of these drainage works. When their capacity is exceeded, the excess often flows overland, causing damage and losses. The purpose of the major drainage system is to accommodate the runoff that exceeds the capacity of the minor system.

The storm drainage system uses the energy available from the difference between its upstream and downstream elevations, or the energy added by pumps. The total available energy normally is used to maintain proper flow velocities, including sufficient velocities to insure a self-cleansing system, with minimum head loss. The design of storm drainage systems therefore involves the balancing of hydraulic losses, which must be kept within the limits of available energy, with the need for adequate energy to maintain self-cleansing velocities. The wider the variations in the rates of flow, the more difficult it is to meet both requirements (ASCE 1969).

This chapter focuses on the design of conveyance facilities common to the major and minor systems, including storm sewers, streets, open channels, culverts, and bridges. Details on their hydraulic design are presented in Chapter 6, and one should be completely familiar with that Chapter (and with Chapter 5, Hydrology and Introduction to Water Quality) before proceeding with any design. All of the facilities described closely interrelate, and major and minor system planning and design should be conducted concurrently.

The designer should treat the design effort as an iterative process involving the developer, regulators, planners and landscape architects, the general project civil engineer, and others (National Association of

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Homebuilders 1991; Urban Land Institute 1975). All of these individuals have much to contribute, and optimum drainage solutions usually are those generated by various parties with different points of view (Spirn 1984; American Society of Civil Engineers 1985).

II. FREQUENCY OF DESIGN RUNOFF

Historically, storm drainage systems were designed solely for the rapid removal of stormwater, and hydrological considerations were largely restricted to sizing conduits (American Society of Civil Engineers 1985). The performance criterion was how effectively the land had been drained. It is common now, however, to require that these systems meet several regulatory criteria, including containment of flows and their pollutant burdens. The designer therefore is faced not only with the problem of assuring that flooding will be minimized, but also that the system will meet established limits on the discharge of pollutants.

The selection of a frequency of design runoff, commonly (and erroneously) called the "design storm," to satisfy the various regulatory criteria must be carefully done. As noted in Chapter 5, it can be easily shown (McPherson 1978; WPCF 1989) that, for a given rainfall event, the return frequencies of peak runoff flow, flow volume and the pollutants associated with that flow will probably be different from each other, as well as different from the return frequency of the rainfall itself.

The normal level of protection expected from the **minor system** ranges from once in two years to once in ten years (used in most cities in North America). Typical return frequencies were presented in Table 5.1. It must be recognized that site-specific conditions can justify deviations from Table 5.1, and "blind" acceptance of these values is not appropriate. As noted in Chapter 1, there has not been, over the years, any adequate rationale developed for the selection of any particular design return frequency for either the minor or the major system.

The return frequency of design runoff used for the **major system** typically ranges from 25- to 100-years, and should be selected based on an evaluation of the specific project under consideration and any local or regulatory requirements. Once the overall design return frequency has been set, the system should be examined for points where deviation may be justified or necessary. If this analysis reveals serious problems with the existing major system, then remedial measures should be considered (Denver Urban Drainage and Flood Control District 1984), such as over-designing the minor system to compensate for deficiencies that may exist in the major system. The marginal economic benefits to be obtained should be calculated, if possible, to provide information for the economic (political) decisions that inevitably follow.

III. GENERALIZED DESIGN PROCEDURES

These procedures apply generally for new drainage systems. For existing systems, one or more of the steps may not be necessary.

A. Preliminary Design

1. Define Project Goals and Objectives

Define the goals and objectives (water quality and quantity) that will guide the project. What does the project proponent want the design to accomplish? What is expected of the design engineer? What is the scope of the assignment? Reference: Chapter 4.

2. Define Pertinent Regulations and Criteria

Identify all applicable federal, state, and local regulations and criteria that will affect the design. For contemplated drainage structures, list criteria that will apply (major/minor system return frequencies, freeboard in channels, pollutant removal requirements, etc.). If local criteria are not available, obtain guidance from local regulators (note that national and/or state criteria may apply). Reference: Chapter 2.

3. Collect Basic Data

Collect background information (soil types, locations of wetlands, existing drainage structures, rainfall characteristics, historic flood information, water quality data, groundwater situation, survey/mapping/GIS data, etc.), including prior studies in the area. Reference: Chapter 3.

4. Determine Limits of Basin and Analyze Preliminary Data

Classify probable future development within the basin as it affects both hydrology and hydraulic design (see Chapter 5). Off-site areas that drain onto the site, not just the site itself, must be included. Classify streets as to storm water drainage carrying capacity. Determine design frequency for initial design. Obtain or develop rainfall data (intensity-duration-frequency curves, design storms, or time-series data, depending on hydrologic methods being used) and calculate associated runoff for both the minor and the major system.

Identify location of discharges (outfalls) for the project, along with their capacity and downstream constraints. Identify natural drainageways through the site.

When sub-dividing the total drainage area of interest, remember that at various inlets on a continuous grade, only a portion of street flow will be diverted to the storm sewer system (due to "carryover"), and at intersections of collector streets or arterial streets it may be necessary to remove 100% of the minor drainage from the road surface to preclude

cross-street flow. The sub-basins should vary according to the actual storm sewer system layout being considered.

5. *Obtain Site Development Plan and Formulate Conceptual Alternatives*

Obtain initial site development plan from the project proponent or planner and formulate conceptual alternative drainage approaches. Based on "back of the envelope" hydrology and field inspection, identify potential "fatal flaws" in the development plan, which can include:

(a) Obliteration of existing major drainageways.
(b) Adverse effects of increased runoff (peak flow/volume).
(c) Site development plan that conflicts with the natural hydrologic characteristics of the site rather than enhances these characteristics.
(d) Lack of attention to whether the site drains to an outfall with assured capacity. Where will the water go once it is discharged off-site and what hazards are posed to downstream properties?
(e) Off-site flows that enter but do not safely pass through the site.
(f) Insufficient attention to local, state or federal permitting requirements.
(g) Failure to set aside sites for detention storage.

It may be desirable to modify the proposed layout to minimize the need for drainage and sanitary sewer facilities, and for earthwork. Conceptual alternatives will vary according to such factors as magnitude and type of detention; major/minor system alignments (flow paths); and channelization to convey full major discharge versus tight land use controls in floodplains.

6. *Refine Conceptual Alternatives to 2–3 Preferred Strategies*

Review the reasonable alternative layouts for both the major and minor systems, selecting those that appear most practical, economical, and consistent with project objectives. A number of iterations between the planner and drainage engineer may be required. The most promising concepts should be selected for analysis.

Existing drainage, whether clearly defined channels or more subtle "swales," should normally be set aside as major drainageways. Rolling, hilly terrain usually has natural drainage patterns that cannot be significantly modified. If existing major drainageways are to be blocked by land development, alternative drainage capacity must be provided. Transferring the problem from one drainage basin to another, however, should be strictly avoided (Denver Urban Drainage and Flood Control District 1984).

The safety, convenience and cost effectiveness of preserving natural drainage to serve as "outfalls" for street and storm sewer discharges are well recognized, and the designer is urged to identify and utilize the natural system to its fullest advantage (Sheaffer et al. 1982; Urban Land Institute 1975).

7. Preliminary Design.

Proceed with preliminary design of the major/minor systems for each of the remaining alternatives. The methods for determining design runoff are discussed in Chapter 5 (see also Section XI, this chapter). Design runoff rates may need to be adjusted to reflect detention requirements, in accordance with the procedures described in Chapter 11.

Preliminary design consists of the following kinds of activities; this list is not meant to be comprehensive and the reader is referred to Chapters 4, 8, 10, and 11 for further detail. These tasks should be conducted for each of the alternatives.

a. Define alignments and grades for storm sewers and open channels. For pipes, identify factors that will influence alignment and grade, such as other utilities (crucial), railroads, tree preservation, embankments, buildings, etc. Other factors that influence channel grade include existing slope (of natural channels), erodibility, available right of way, and channel lining. Several preliminary layouts may be considered. Pipe alignments should probably coincide with collector or arterial streets, since it usually will be necessary to preclude cross street flow on these streets. Experiment with different approaches to optimize the number and size of storm water conveyance structures required.
b. Define detention strategy and storage locations, how detention will influence the outfall system for each alternative (APWA 1981), and the role detention storage will have in water quality enhancement (Schueler 1987).
c. Define the inception points of storm sewers—the locations where specified street carrying capacity is exceeded. Examine ways to reduce flows in headwater areas so as to be able to move the starting point of storm sewers downstream (to save money).

   Preliminary street grades and cross-sections must be available to the designer. Beginning at the upper end of the basin in question, calculate the flow in the street until the allowable street carrying capacity matches the design runoff. The storm sewer system will begin at this location if there is no alternate method of removing runoff from the street surface (Malcom 1989). It is not necessary to remove all of the flow from the street surface at the beginning of the storm sewer system, or at any given location along the system, unless the intersection of streets dictates that no cross-street flow may occur. It is, however, necessary that the allowable street capacity plus the storm sewer capacity equal or exceed the design flow.

   This process should account for the fact that some portion of the total runoff will not reach the streets, but will instead be conveyed through natural drainage which is unaltered by development, by man-made open channels, or simply by "inadvertent" grading of yards and open spaces.

   The portion of flow removed from the street surface becomes the design flow for the storm sewer. For preliminary design purposes, a Manning's "n" value, or other roughness coefficient, about 25% above that contemplated for final design should be used in calculation. Using

this n value, the preliminary grades established earlier, and assuming the sewer is flowing full, a pipe size for the design flow may be determined from an applicable chart or formula, or by using one of the many commercially-available computer programs developed for this purpose. This method for sizing pipes should be adequate for preliminary design purposes, since the increased roughness coefficient offsets the effect of minor losses. Flow velocities will be sufficiently accurate for routing calculations. Cost estimates can then be based on these sizes and assumed depths of cut.

d. Starting in the study area headwaters and proceeding to the downstream point of interest, calculate flows at ultimate development, using the major system design runoff, in the major drainageways, and determine the right-of-way required to convey the flows (with an allowance for freeboard).

   Determine if the combined capacity of the street and storm sewer system is sufficient to maintain surface flows within acceptable limits during the major storm. The combined total of the allowable street carrying capacity for the major storm and the storm sewer capacity should equal the major design runoff. At any given point along the storm sewer system, the capacity of the sewer can usually be assumed to be the same for the major runoff as for the minor runoff, unless special considerations indicate otherwise.

   In the uppermost area of a drainage basin, major drainageways may not consist of readily distinguishable channels—they may be homeowners yards, swales or streets or whatever "low ground" exists—and for the entire study area, the designer should determine the path that the major system design runoff event will take. This land should be set aside as a perpetual "flow easement." Preliminary design-level channel sizing can normally be done via hand calculations using normal depth techniques and assumptions regarding location of hydraulic "controls."

e. Define all hydrologic "design points" for channel and storm sewer reaches, including drainageway confluences, street intersections, detention storage locations, junction structures, culverts, etc. When establishing the design points, inspect future condition site mapping to establish flow paths for all parcels on the site.

f. Conduct detailed hydrologic computations to quantify design flows at all drainage structures of interest.
   1. Compute off-site and on-site flows.
   2. Calculate ultimate development flows and "historic" flows for floods ranging from the 2-year event to the major system design runoff. Under particular circumstances, it may be necessary to consider runoff events smaller than the 2-year (for water quality design purposes) or larger than the major system design runoff.

      If it is impractical to accommodate the major system design flows using the combination of the street and storm sewer carrying capacity, the engineer has these options: (1) change the major system design basis (which may require an important policy decision), (2) increase the minor system capacity, (3) elevate or otherwise protect buildings, (4) begin a formal major drainageway, or (5) provide upstream detention to decrease downstream discharge. Each of these choices will have economic implications.

The urban basin size required to effectively mandate that a formal major drainageway be reserved varies from location to location. In Denver, Colorado, for example, policy states that a properly designed, engineered and maintained open channel (an "outfall") will be established for all urban basins larger than 130 acres.

3. Based on the analysis in f. 2, applicable local criteria, and interviews with regulators, establish whether or not stormwater detention is required. Size detention facilities in accordance with the procedures described in Chapters 5 and 11.

g. Given a storm drainage system alignment and estimates of runoff discharges for the specified minor system return frequency, locate and size inlets. Pay particular attention to intersections where carryover is not acceptable.

h. Given storm drainage system alignment, grades, and inflows (via inlets), the hydraulic grade line and energy grade line for the storm sewer should be computed. Adjust alignment and grade as required to comply with criteria. Assure that the hydraulic grade line remains a specified distance below the ground surface for the minor event.

i. Not all of the minor system will be below ground. The major drainageways and associated hydraulic structures (culverts, transitions, etc.), surface swales and detention facilities will also need to function properly during the minor storm. Accordingly, evaluate their behavior during the minor event.

j. Check the function of the overall minor system during the design event and assure that adequate outfall capacity is available.

k. Given designated major drainageways (with right-of-way set aside to accommodate the design runoff) and a functioning minor system, evaluate where the flood waters, in excess of those being conveyed in the storm sewer, will flow. Also evaluate how site topography must be adjusted to assure that these flows will be conveyed safely downgradient without threatening lives, safety, or property.

l. Evaluate behavior of detention facilities during the major system design flow condition. Assure that detention dams, which could pose a threat to human safety or property if they fail, are designed to handle flows larger than major system design flows (Sheaffer et al. 1982).

m. Special evaluation of culverts and bridges, in accordance with the procedures described later in this chapter, will be required. Assure that applicable criteria are not violated. Adjust the size and characteristics of the conveyances until the amount of backwater during the major system design runoff event meets target levels. Define the area affected by backwater flooding during the major event and indicate that this land should remain undeveloped for perpetuity (Jones and Jones 1987).

n. Define right-of-way requirements for alternatives.

o. Prepare preliminary-design level capital and operation/maintenance costs for the alternatives. If feasible and appropriate, use life cycle cost theory (see discussion later in this chapter) for economic evaluation (Urban Drainage and Flood Control District 1987). The design cost objective should be to minimize the total annual costs of the drainage facilities and flood-related damage.

q. Evaluate the alternatives with respect to important qualitative criteria such as preservation of open space, enhanced wildlife habitat, impact on wetlands and water quality benefits (if no formal attempt

has been made to contrast the pollutant removal capabilities of various alternatives).

r. Prepare a preliminary design report that contrasts the alternatives quantitatively and qualitatively (in a form suitable for submission to the client and regulators).

8. *Review Alternates and Formulate Preferred Alternative for Final Design*

Meet with client, project planner, general project civil engineer and others to review preliminary design work and to select a preferred alternative. Unbiased evaluation of the good and bad points of each alternative is necessary if the most desirable system is to be identified. Quantitative and qualitative factors should be assessed. If any potential problems exist, such as effects on downstream entities, large conflicts with utilities, or difficulty in acquiring easements, they should be thoroughly reviewed and resolved.

Adjust the preferred alternatives (still at the preliminary-design level) to accommodate the requests of the client and his advisors. Submit preferred preliminary design to regulators. Adjust preliminary design as required to fulfil regulatory requirements and obtain approval of the preliminary design.

B. Overview of Final Design

The preceding steps constitute the preliminary design effort. Up to this point, the engineer has been compiling the information necessary to make an informed decision on which system to use for final design (Urban Drainage and Flood Control District 1984). The following steps will complete the process.

1. *Review all Preliminary Work*

Hydrologic assumptions, basin boundaries, sub-basin delineations, street classifications, assumed/calculated pollutant removal efficiencies, and any other preliminary design values that will be used subsequently in final design should be reviewed for accuracy and applicability to final design. Unresolved questions must be answered at this time, and the designer should step back and ask himself if his design constitutes sound engineering practice. Fundamental compatibility of site layout with drainage needs must be assured at this stage (Urban Land Institute 1975).

2. *Obtain Final Street Grades, Geometry, Elevations, Etc.*

Often it will be necessary to revise street construction details to facilitate drainage. This may include eliminating cross fall on streets,

raising required ground elevation at buildings adjacent to streets to accommodate major drainage, or increasing street gradients to achieve sufficient capacity within the street. It is especially important to assure that first floor elevations of buildings are well above street crown elevations to prevent repeated flooding (Jones 1980).

3. *Hydraulically Design the Open Channel and Storm Sewer Systems*

The final hydraulic design of a system should be on the basis of procedures set forth later in this Chapter and in Chapter 6. A realistic "n" value for final design should be determined and applied, treating the conduits as either open channels or pipes flowing full, as appropriate. For open channel flow, the energy grade line should be used as the base for calculation. For conduits flowing full, the hydraulic grade line should also be calculated. The energy grade line governs for major drainageways.

For storm sewers, the design engineer must review the hydraulic grade line for various runoff conditions (initial design runoff and others that are larger) to insure that the hydraulic grade line is consistent with desired system performance.

4. *Complete all Other Aspects of the Design Effort*

The final design should address all other factors, including structural and geotechnical design, permitting, water quality, construction documents, coordination with other aspects of the project, etc.

IV. SYSTEM LAYOUT

Design begins with system layout—approximately defining the minor and major flow routes. System layout includes the selection of an outfall, delimiting drainage area boundaries, and identifying the locations of trunk and main sewers, or outfall channels that will feed the outfall. Initial layouts can usually be done from topographic maps.

The ease with which the designer will be able to locate and utilize an "outfall" to discharge into will vary widely depending principally upon the extent of pre-existing development and whether the overall basin drainage was properly planned to assure that adequate outfall capacity would be available as development occurs. An inadequate outfall may force the design towards detention, whereas an oversized outfall could result in reduced on-site detention requirements. The layout of the minor system should generally conform to the following standards, which will vary from locale to locale.

A. Location Requirements

1. *Main Location*

Storm sewers are normally located a short distance behind the curb, or in the roadway near the curb. On occasion, it may be necessary to locate them in right-of-way easements, but locating them on public property is preferred.

2. *Alignment*

Storm sewers should be straight between manholes where possible (ASCE 1969). Where curves are necessary to conform to street layout, the radius of curvature should not be less than 100 feet. Radius of curvature specified should coincide with standard curves available for the type of material being utilized. Short radius curves at manholes can be designed per Table 8.1. Short radius bends are not recommended on sewers 21 inches or less in diameter.

3. *Crossings*

Crossings with other underground utilities should be avoided whenever possible, but, if necessary, should be at an angle greater than 45 degrees. Utility crossings are a major design factor (and cost) when retrofitting in urban areas, and considerable effort is required to comprehensively define and locate utilities in the field.

B. Manholes (Cleanout Structures)

Manholes are normally located at the junctions of sewers and often at changes in grade or alignment. Manhole spacing is usually specified locally, but Table 8.2 provides general values. Short radius bends are

TABLE 8.1. Typical Allowable Radius of Curvature for Short Radius Curves (Boulder County 1984).

Diameter of Pipe (1)	Minimum Radius of Curvature (2)
24" to 54"	28.50 ft.
57 to 72"	32.00 ft.
78" to 108"	38.00 ft.

Note: Short radius bends should rarely be necessary because pipe alignments usually follow street alignments. In the rare situation where a short radius bend is necessary, discharge energy and geotechnical conditions should be evaluated to determine need for thrust blocks.

TABLE 8.2. Typical Manhole Spacing (Boulder County 1984).

Pipe Size (1)	Maximum Spacing (2)
15″ or less	400 feet
18″ to 36″	500 feet
42″ or greater	600 feet

Note: With pipes larger than 24″–30″, manholes usually may be spaced as far apart as possible, consistent with sewer cleaning limitations.

often used on 24″ and larger pipes when flow must undergo a direction change at a junction. Reductions in head losses at manholes may be realized in this way. A manhole should always be located at the end of such short radius bends.

C. Grade and Cover

The minimum slope should be capable of producing a velocity of at least 2–3 feet per second when the sewer is flowing full. Consideration should be given to the capacity required, sedimentation problems, and other design parameters.

The sewer grade should assure that there is a minimum of 3 feet of cover over the crown of the pipe (depending on local practice). Uniform slope between manholes is desirable for ease of maintenance. The sewer system must be structurally reinforced if sufficient vertical clearance (cover) is not available. Railroad and highway crossings pose a particular concern. Structural loading calculations are recommended for all such installations. For further structural details, see Chapter 14.

V. HYDRAULIC DESIGN OF STORM SEWERS

A. Design of Closed Conduits

The following design procedure is applicable to closed conduits flowing with a free water surface. Although it is theoretically possible for a pipe flowing less than full to carry more than when it is full, it is impractical and imprudent to assume that this larger conveyance will be available.

1. *Range of Applicability*

The following design procedure is based on the assumption of a uniform hydraulic gradient within pipe reaches. Where conduits are sufficiently large, or a higher degree of accuracy is needed, water surface profiles can be calculated (see Chapter 6 for water surface profile computations). In local drainage design, however, the computation of water

surface profiles will rarely be required as flows and conveyance capacities are small (Urban Drainage and Flood Control District 1984).

2. *Design Procedures*

The basic approach to design of closed conduits consists of calculating the energy grade line along the system profile. In most situations, one can assume that the energy grade line is parallel to the pipe grade, and any losses other than pipe friction may be accounted for by assuming point losses at each manhole (this assumption is not valid, however, whenever a hydraulic jump is likely to occur within a pipe).

Once the discharge has been determined, and a pipe size and slope assumed for a given section, the d/D and v/V_{full}, ratios can be determined from a graph of hydraulic elements and the energy grade line calculated as described in Chapter 6.

At each manhole the energy grade lines of all pipes should coincide, allowing reasonable losses at the junction.

3. *Losses at Manholes*

Methods for calculating losses in manholes were described in Chapter 6. Recent studies by Marsalek and the American Public Works Association (Marsalek 1986, 1987, 1988) have shown that properly designed bottom sections can reduce manhole losses significantly. It should be noted that it is common to find large storm drainage conduits converging at a junction at an acute angle. Such junctions should be carefully designed (see, for instance, the City of Los Angeles 1968) to avoid large head losses.

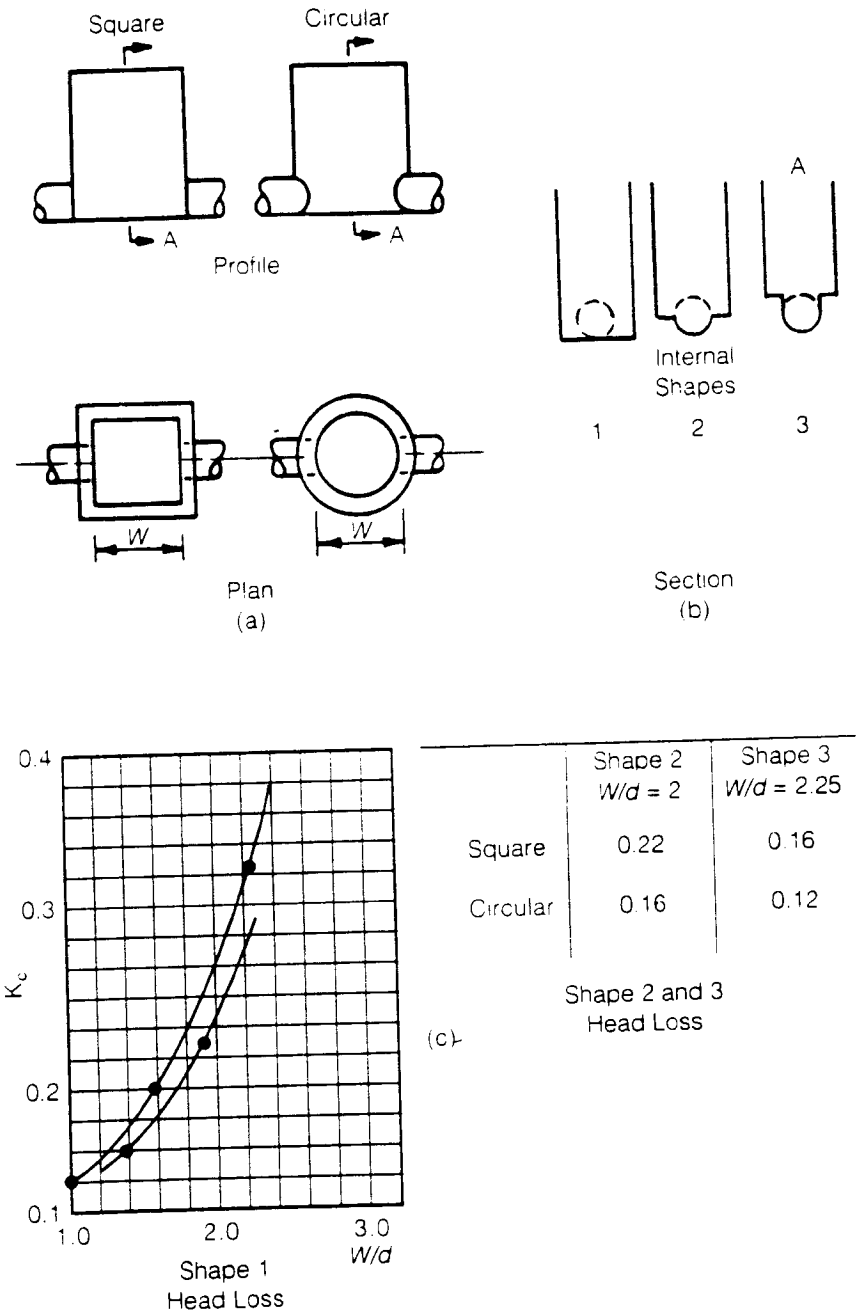
Clean-Out or Manhole Structures with Straight Flow Flow goes straight through the structure, which may be square or circular in plan (Figure 8.1a). The pipe diameter is the same on both the inlet and outlet side, with several internal shapes (Figure 8.1b) possible. Shape Types 2 and 3 have a bench constructed in the bottom of the manhole to improve flow paths. In Type 2, the bench is at the centerline of the pipe, and in Type 3, the bench is at the top of the pipe.

Head losses can be calculated using Equation (6-33). Values of K_c vary with the ratio of structure width (W) to pipe diameter (d), and are given in Figure 8.1c. Head losses in Types 2 and 3, which are benched, are less than those in Type 1.

Clean-Out Structures at 90° Bends Head loss coefficients for Equation (6-33) in structures at 90° bends (with and without benching) are shown in Figure 8.2.

Coefficients are for fully submerged flow, and open-channel flow coefficients are about ⅓ less.

Alignment of Pipe in Manholes For straight-through flow, aligning the inverts of the pipes is generally advantageous as the manhole bot-



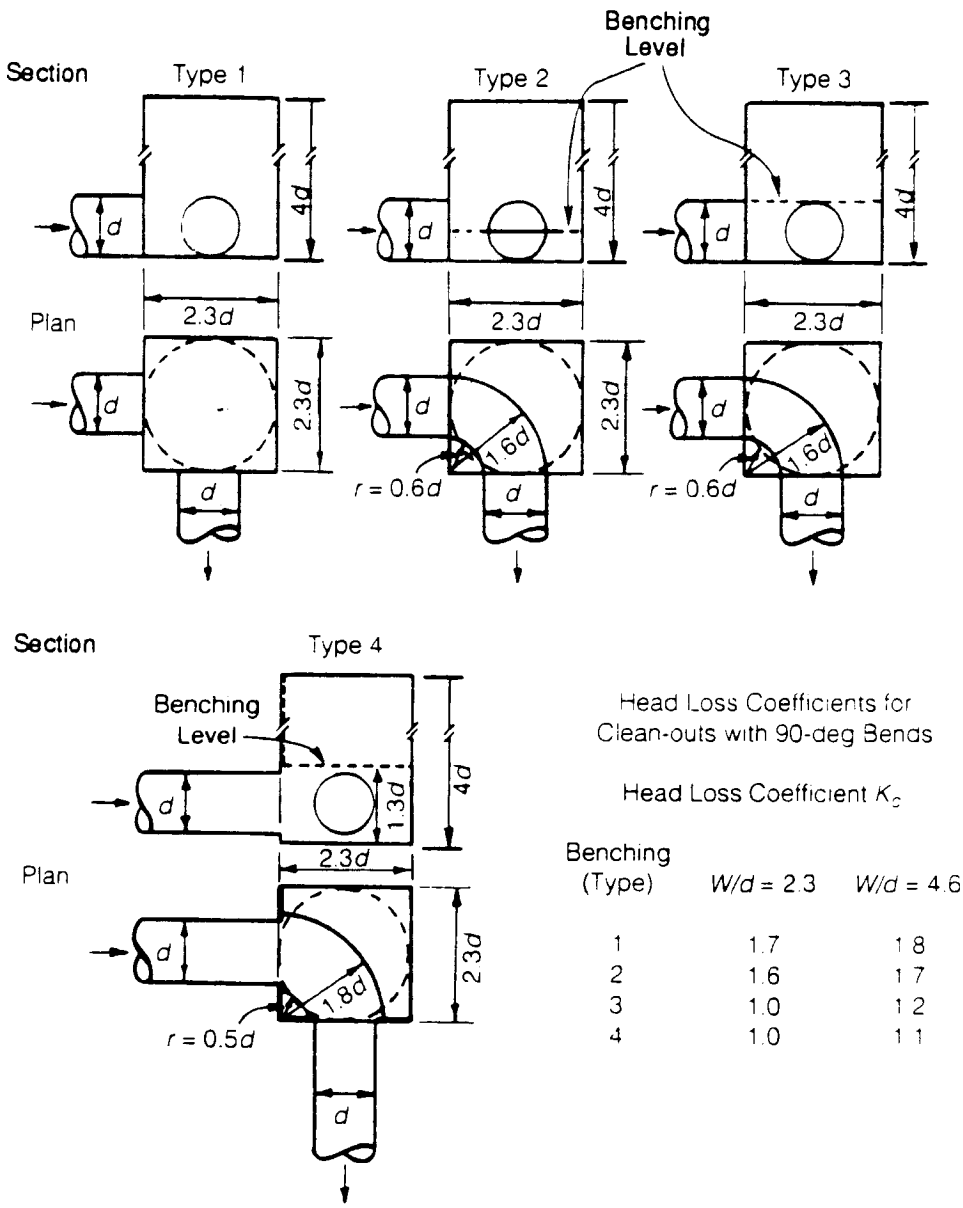
	Shape 2 $W/d = 2$	Shape 3 $W/d = 2.25$
Square	0.22	0.16
Circular	0.16	0.12

Shape 2 and 3
Head Loss

Figure 8.1—Head loss coefficients for internal shapes: (a) profile and plan, (b) internal shape (section), and (c) head loss coefficients for internal shape 1 curve and internal shapes 2 and 3 (Marsalek, 1985).

tom then supports the bottom of the jet issuing from the upstream pipe.

When two laterals intersect a manhole, the alignment should be quite different. If lateral pipes are aligned opposite one another so the jets may impinge upon each other, losses are extremely high. If directly



Head Loss Coefficients for Clean-outs with 90-deg Bends

Head Loss Coefficient K_c

Benching (Type)	$W/d = 2.3$	$W/d = 4.6$
1	1.7	1.8
2	1.6	1.7
3	1.0	1.2
4	1.0	1.1

Figure 8.2—Clean-out structures (Marsalek, 1985).

opposed laterals are necessary, the installation of a deflector as shown in Figure 8.3 will result in significantly reduced losses. Some examples of inefficient manhole shapes are provided in Figure 8.4.

Entrances Rounding entrances or using pipe socket entrances to provide smooth transitions between pipe and manhole will reduce losses, though installation costs may be high in retrofitted systems.

B. Pressurized Storm Sewers

Storm sewers are often planned as pressure conduits for the design runoff, or to operate under pressure when runoff occurs that is greater

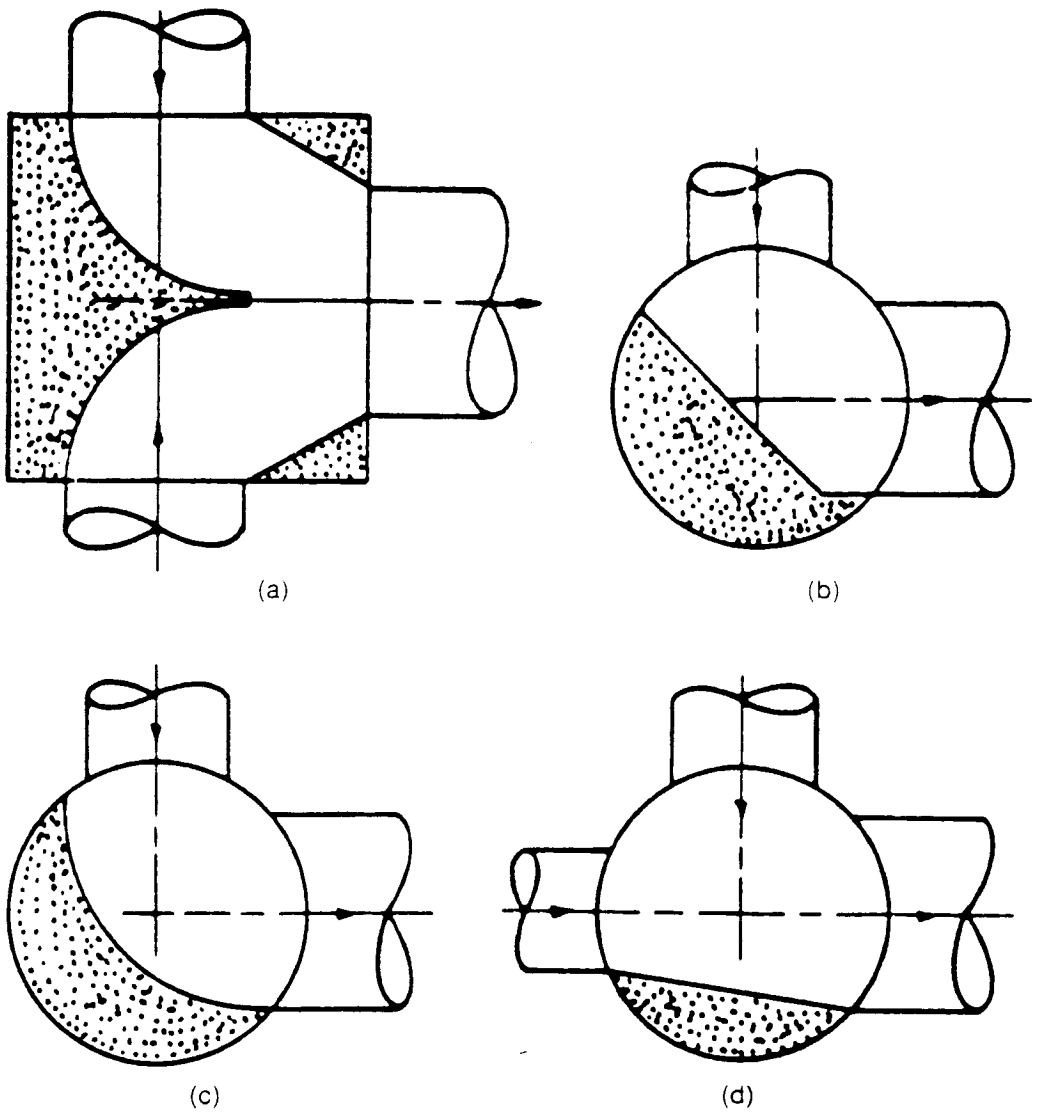


Figure 8.3—Efficient manhole shaping: (a) directly opposed lateral with deflector (head losses are still excessive with this method but are significantly less than when no deflector exists), (b) bend with straight deflector, (c) bend with curved deflector, and (d) inline upstream main and 90-deg lateral with deflector (Urban Drainage and Flood Control District, 1984).

than design flow. The fact that such storm sewers usually have manhole and storm inlet appurtenances that provide a direct hydraulic connection to the street surface means that special care must be taken in their analysis.

If the hydraulic grade line rises above the ground surface, storm inlets will not function, and storm water will be discharged from the storm sewer to the street surface via the inlets and manholes. Overdesign of upstream storm inlets, which is encouraged as good practice, can lead to this situation (a frequent cause of "popping" manhole

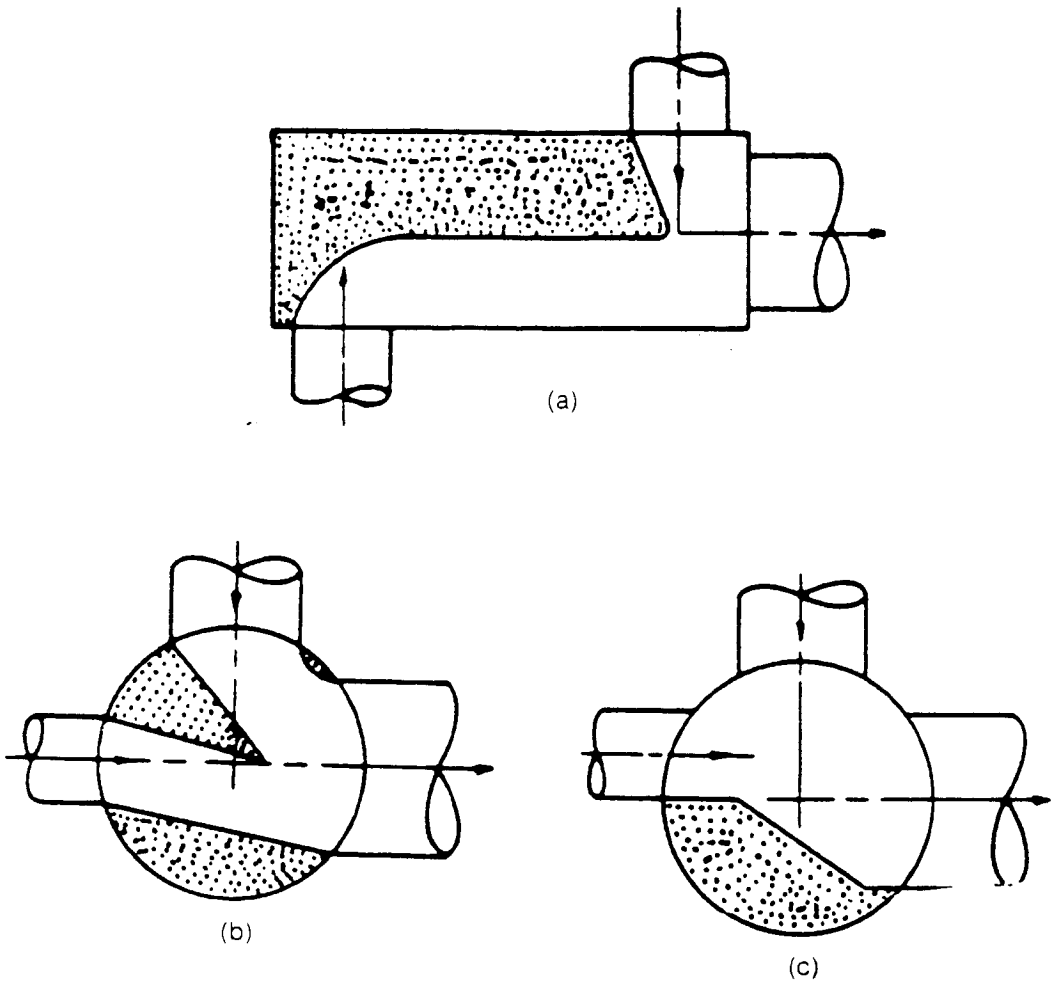


Figure 8.4—Inefficient manhole shaping: (a) offset lateral with deflector, (b) inline upstream main and 90-deg lateral with divider, and (c) inline upstream main and 90-deg lateral with deflector (Urban Drainage and Flood Control District, 1984).

covers). Sometimes manholes with anchored and sealed covers are designed for a hydraulic gradient higher than street level with limited storm inlet connections.

The following design procedures are applicable where it is found advisable or necessary to have storm sewers flow full as pressure conduits. Checks must continually be made to verify if the conduit is in fact flowing full. Often the storm sewer system will alternate between pressurized and open channel flow. In such cases, it will be necessary to establish the type of flow and design accordingly.

1. Basic Design Procedures

The basic design procedure for a pressurized storm sewer consists of: (1) determining allowable pressures, (2) determining type of flow,

(3) computing pipe junction and form losses, (4) computing the hydraulic grade line, and (5) integrating minor and major system behavior during the major system design runoff event.

The initial sizing of a pressurized system typically begins in the upstream portion of the basin, where the above-ground flow capacity is exceeded and underground pipes are needed. The design process proceeds in a downstream direction. After initial sizing, the system is checked for adequacy by calculating the hydraulic and energy grade lines, beginning at the downstream end of the system (i.e. the receiving water surface) (Clark et al. 1977). All this information is plotted on a profile that shows the pipe, the ground surface, utility crossings, etc. This process may require adjusting pipe sizes until a desired hydraulic grade line is achieved.

2. *Allowable Pressures*

Two major considerations limit the maximum allowable pressure in a sewer. First, the structural limitations must not be exceeded for a given pipe (this rarely is the controlling factor). When considering structural limitations, both the pipe and the joint must be analyzed. Second, the hydraulic grade line should be kept below ground level (or below the basement floor level, for buildings connected by drains to the sewer system) for the design runoff event, unless special precautions are taken to prevent water from escaping from inlets and manholes, or to handle it once it does escape. A further limitation is to assure a sufficient drop into inlets to allow them to function properly.

3. *Discharge Point*

The discharge point of the system usually establishes a control point. If the discharge is submerged, as when the receiving water level is above the crown of the sewer, the exit loss should be added to the hydraulic grade line and calculations for head loss in the system started from this point, as illustrated in Figure 8.5. If the hydraulic grade line is above the pipe crown at the next upstream manhole, full flow calculations may proceed. If the hydraulic gradient is below the pipe crown at the upstream manhole, then open channel flow calculations must be used at the manhole.

When the discharge is not submerged, a flow depth must be determined at some control section to allow calculations to proceed upstream. As shown in Figure 8.5, the hydraulic grade line is then projected to the upstream manhole or inlet. Full flow calculations may be used at the manhole if the hydraulic gradient is above the pipe crown.

Urbonas and Roesner (1991) provide a good discussion of how coastal (tidal) variability and lake level fluctuation affect the selection of starting water surface elevations. Regardless of the point of discharge, however, the designer must define the conditions likely to prevail under "design" flows.

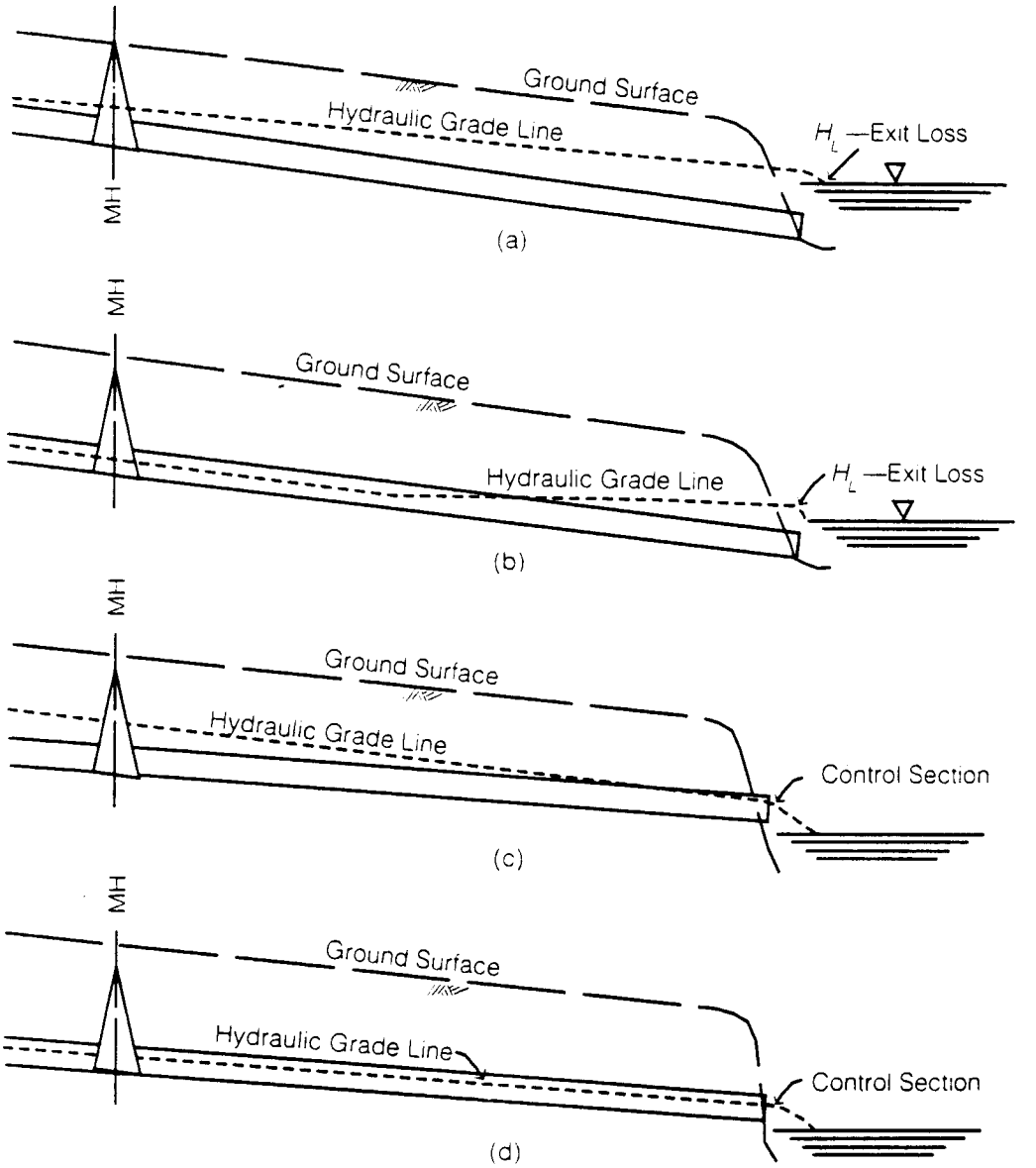


Figure 8.5—Determining type of flow: (a) submerged discharge—hydraulic grade line above crown of pipe, full-flow design methods may be used at manhole; (b) submerged discharge—hydraulic grade line below crown of pipe, open channel flow methods must be used at manhole; (c) free discharge—hydraulic grade line above crown of pipe, full-flow design methods may be used at manhole; and (d) free discharge—hydraulic grade line below crown of pipe, open channel flow methods must be used at manhole (Urban Drainage and Flood Control District, 1984).

4. *Within the System*

At each manhole, a procedure similar to that outlined for the discharge point must be repeated. The water depth in each manhole is calculated to verify that the water level is above the crown of all pipes.

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Whenever the level is below the crown of a pipe full flow methods are not applicable.

5. *Friction and Form Losses*

Friction and form losses should be calculated in accordance with the methods outlined in Chapter 6.

VI. STORM SEWER INLETS

A. Categories, Definitions, and Applications

Inlets are used to intercept and convey surface runoff to closed conduit hydraulic systems. When used in streets, inlets are generally placed in the street curb and gutter. Gutter inlets can be classified as one of four types: curb; grate, or yard opening; curb and grate (combination); or special purpose inlets. Gutter inlets can be placed on a continuous grade, a partially continuous grade with a local depression, or in a sump. Several combinations of these inlet types are illustrated in Chapter 6, which includes a detailed discussion of inlet hydraulics.

The hydraulic capacity of a gutter inlet depends upon inlet geometry (the capacity of which can be affected by expected debris loadings and maintenance), and the characteristics of gutter flow (which can be a function of roadway configuration).

Inlets placed on a continuous grade are usually designed to allow runoff bypass. For a given design-flow, gutter inlet capacity can be increased by using a gutter/inlet opening depression, steeper street cross slope, and/or a flatter longitudinal street slope. Inlet capacity can be increased at locations with a local sump condition where a certain amount of gutter ponding is allowed before runoff spills and is conveyed downstream. Length, width, depth and slope of a local depression have significant effects on the capacity of an inlet. Finally, inlets can be installed in a sump, where ponding is allowed to a regulated depth. The effect of a local depression on inlet capacity is graphically illustrated in Figure 8.6 (Johns Hopkins University 1956). Inlet capacity increases for both grate and curb opening inlets with local depressions on flatter longitudinal grades.

Reduction factors are generally used to reduce theoretical hydraulic inlet capacities to account for potential debris blockage and future street resurfacing. Reduction factors are usually obtained from local criteria manuals. The efficiency of an inlet is defined as the ratio of intercepted runoff divided by the total runoff in the upstream roadway cross-section. During inlet design, overflow paths should be identified at proposed inlet locations, especially where a sump condition will occur. This precaution will lessen the chance for incurring damages when runoff exceeds the inlet capacity or if debris blockage should occur.

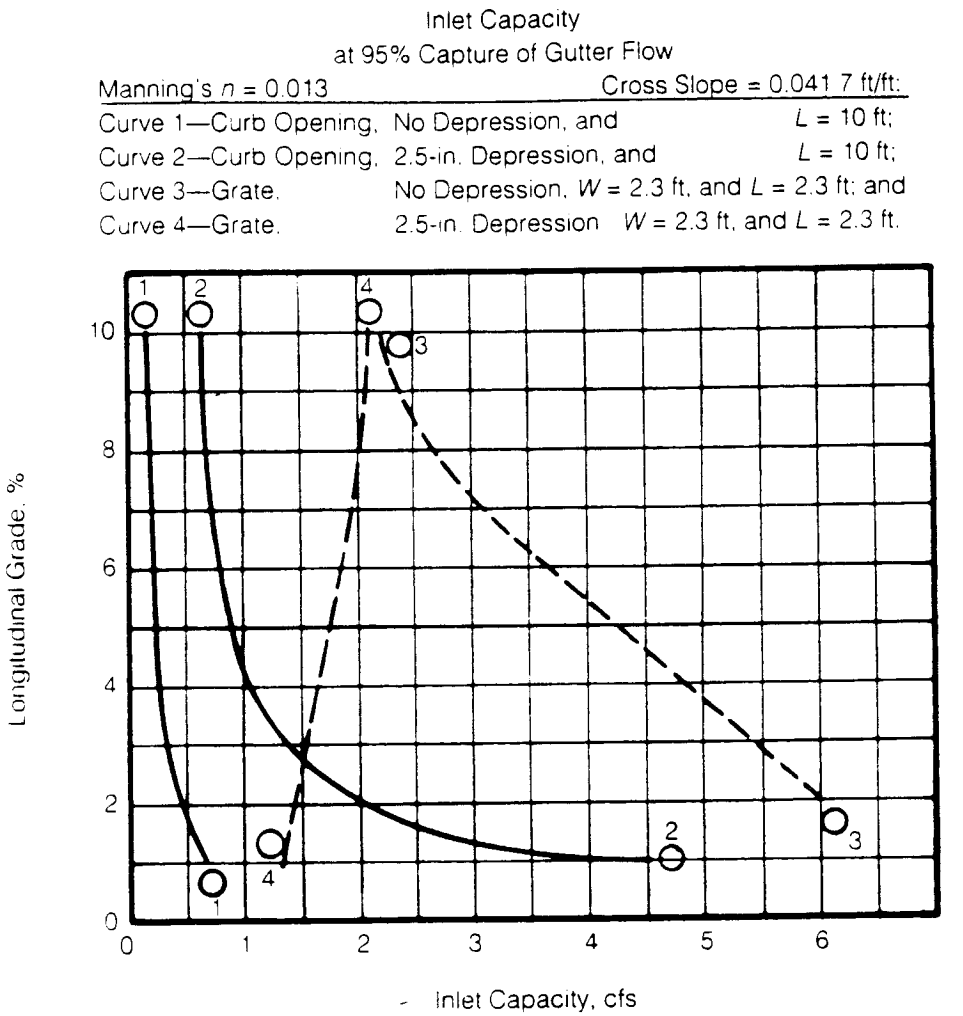

Figure 8.6—Comparison of inlets with and without local depressions (cfs × 0.028 32 = m^3/s, ft/ft × 0.304 8 = m, and in. × 25.4 = mm) (Johns Hopkins Univ., 1956).

B. Curb Opening Inlets

Curb opening inlets are most effective, and have several advantages over grate- and combination-type inlets. They do not encroach into the travelled way of either motorized vehicles or bicycles. Where there is pedestrian access, curb opening inlets are safer than other types of inlets. They are also more effective than other types of inlets where debris flows are expected. Opening heights should not exceed 6", to reduce risks to children.

A disadvantage of curb opening inlets is their sensitivity to changes in street grade, both along continuous grades and at local depressions. Curb inlets, like other types of inlets, lose capacity both with steeper longitudinal street slopes and flatter street cross slopes. Generally, curb opening inlets on continuous grades without a local depression are very inefficient. Their capacity and efficiency can be estimated using methods

presented in Chapter 6. Additional information can be found in various publications (Izzard 1977; Bauler and Woo 1964; U. S. Department of Transportation 1972).

C. Grate Inlets

Grated inlets are less sensitive to street grade changes than curb opening inlets. Disadvantages of grated inlets include their interference with motorized vehicles and bicycles and their tendency for debris blockage, with resultant loss of capacity. Some cities have discontinued their use for these reasons.

The capacity of non-depressed grated inlets on continuous street grades can be estimated from published equations and/or charts. If the grate is long enough, the flow intercepted by the grate will consist of the runoff flowing within the width of the grate and possibly a portion of the runoff flow from the street along the length of the grate. An efficient grate has a bar arrangement that does not cause splashing and resultant flow bypass, and that provides a sufficient area of opening to accommodate the design flow.

A safety factor should be applied to theoretical grate inlet capacities, especially if the inlet does not have a curb opening (combination inlet).

D. Curb and Grate (Combination) Inlets

Curb and grate inlets offer the advantages of both curb inlets and gutter inlets. Combination inlets are frequently used in a sump location and provide a greater capacity than the curb or grate type inlet alone.

E. Special Purpose Inlets

1. Bicycle-Safe Grates

Studies by the FHWA (U. S. Department of Transportation 1978) have developed bicycle-safe grate configurations. These studies investigated the hydraulic, structural and debris handling characteristics of seven different inlets, each of which was placed at grade with the gutter pavement. The study results compare hydraulic characteristics of these types of inlets and provide equations for estimating hydraulic capacities and efficiencies.

2. Pipe Drop Inlets

Pipe drop inlets consist of a grate flush with the adjacent ground surface, which drains into a vertical section of concrete or corrugated metal pipe. This type of inlet is not designed for use within roadways. Since a pipe drop inlet has a round surface area, it can intercept flow from any direction, and is most effective for flows that are deepest at the grate center. Generally, pipe drop inlets are economical. They should not be used where vandals may remove the grate, leaving a hazardous open pipe.

3. *Slotted Drain Inlets*

Slotted drain inlets consist of a corrugated steel pipe cut along the longitudinal axis and reinforced with a grate of solid spacer bars. Slotted drains are often used to intercept overland sheet flow from wide, flat areas, but can also be used in a typical curb and gutter. Design nomographs and criteria are available.

F. Inlet Design Criteria and Practices

Storm sewer inlet criteria are most often available from the entity responsible for the formal review and approval of design submittals. Common governmental sources of inlet criteria include the federal government (Federal Highway Administration), the state (Department of Highways), the county, and the township/city. Intergovernmental organizations, such as drainage and flood control districts, often develop broad criteria for those entities within the district's jurisdiction. Finally, criteria are often available from professional and technical society publications (American Society of Civil Engineers), manufacturer catalogs, and organizations with technical programs related to a particular material (American Iron and Steel Institute, American Concrete Pipe Association).

Location and Spacing The placement of curb inlets is generally a trial-and-error procedure since both inlet size and spacing can be varied. The goal in the placement of inlets is the most hydraulically effective system at the most economical cost. The following general rules apply to inlet location design:

(a) Intersections—As discussed earlier, inlets are normally required at intersections to intercept 100% of runoff. This is necessary to prevent street cross flow, which could cause a traffic hazard. Inlets should generally be placed on tangent curb sections and near corners.
(b) Superelevation Transitions—Inlets are generally required where the street cross slope begins to superelevate. These inlets reduce the traffic hazard of street cross flow to the opposite side of the street.
(c) Side Drainage Entrances—Inlets should be placed downstream from points where side drainage enters streets and overloads gutter capacity. Side drainage often results from parking lots. A better practice is to intercept side drainage before it enters the street.

As discussed previously in this section, an inlet is required at the uppermost point in a street where gutter capacity criteria are violated. The inlet location is established by a trial-and-error procedure by which the drainage area is adjusted until a point is located where the gutter capacity is exceeded. The proposed inlet is then sized to intercept a portion of the total flow. Although the references listed previously contain tables summarizing interception capability, 70–80% is a reasonable rule of thumb for flow interception (Urban Drainage and Flood Control District 1984). Succeeding inlets are designed similarly by locating the point where the sum of bypassing flow and flow from the

intervening street (and side drainage, if applicable) exceed the gutter capacity. Inlets should not be placed in driveways, or directly in front of store fronts or private residences.

Finally, the designer must assure that there is an adequate number of inlets to fully utilize the available capacity of the storm sewer.

G. Inlet Selection

When selecting inlets, hydraulic considerations are sometimes sacrificed relative to potential for clogging, nuisance to traffic, convenience, safety and cost. The following factors should be considered before choosing an inlet type:

(a) Likelihood of Clogging—If clogging due to debris is not expected, a grate or combination type inlet will provide more capacity than a curb opening inlet. Otherwise, a curb opening inlet may be favorable. A local depression at either inlet location will increase the inlet capacity.
(b) Traffic Considerations—If traffic is expected close to the curb and the street slope is steep, use a deflector inlet. If the street slope is relatively flat and a potential exists for debris clogging the deflector slots, use a gutter or combination having a grate with longitudinal bars only (these may be hazardous to bicycle traffic).
(c) Safety—This includes traffic and pedestrian safety (affected by the spread of flows around the inlet), as well as safety for bicyclists (Note that attempts to increase inlet efficiency by "dropping" the gutter upslope from the inlet can be dangerous to cyclists).

VII. INFILTRATION AND EXFILTRATION

Infiltration and exfiltration should always be considered during storm sewer design. There are two types: (1) that which is deliberately provided, and (2) that which is unanticipated and which may lead to grave difficulties. See also relevant portions of Chapter 12 for a further discussion.

A. Deliberate Infiltration/Exfiltration

Deliberate infiltration into a storm sewer may sometimes be useful in de-watering areas having persistent or intermittent high groundwater table elevations. It typically is accomplished by installing storm sewers with open or unsealed joints, bedded in granular materials, sometimes even using perforated pipe. Pipe materials selected should be compatible with corrosive and erosive potentials of the groundwater. Total opening area into pipes should be small so that the pipe will primarily serve to convey stormwater runoff and only secondarily, when runoff is negligible, provide an outlet for infiltrating groundwaters. To avoid undesirable groundwater recharge during runoff events, openings for infiltration into a pipe should be limited, consistent with the need to draw down the water table.

When infiltration is desired, the pipe normally should be bedded in granular materials wrapped in a permeable geotextile to provide a granular shell thickness of at least three inches around the pipe. Alternatively, graded granular materials might be provided as bedding, with greater thickness, to filter out soil fines and prevent their conveyance into the pipe. Gradation and cohesiveness of the natural in-place materials will indicate the extent to which their piping may be a potential problem and the extent to which anti-piping measures are appropriate.

Deliberate infiltration should not be used in soils that are expansive (montmorillonite clays), collapsible (naturally cemented loess), or dispersive (some sensitive clays) because of resultant potentially damaging soil movements. Similarly, deliberate infiltration either should not be used or should be used only as a calculated risk where tree or shrub roots might penetrate pipe openings and cause potential blockage.

A special case of deliberate infiltration is where facility perimeter drains or underdrains may be used to remove groundwaters to prevent their adverse effects upon expansive or collapsible soils. Such drains are often bedded in gravel or crushed stone, which requires accumulation of free water in the bedding before outflow can occur to the drain. This accumulation should particularly be avoided in the indicated soils. This involves bedding the drain pipes in lean but relatively impermeable concrete to the depth of openings in pipes, above the pipe invert, so that water cannot accumulate beneath the drain. Where such piping is used for perimeter drains, to help exclude water from beneath facilities, it may be desirable to line the facility side of the drain trench with an impermeable membrane, usually extended beneath the concrete bedding, before backfilling with gravel or crushed stone to within two feet of the ground surface; remaining backfill then should be an expansive clay or other relatively impermeable material, with ground surface grading that will assure rapid surface drainage.

Deliberate exfiltration is a technique that has proven useful for disposal of stormwater runoff from urban impervious surfaces, or to reduce runoff pollutant loadings. Where used, natural soils should be highly permeable, and the maximum potential level of the groundwater table should be no higher than about 2–4 feet (varies substantially, depending on the literature consulted) below the invert of the storm sewer pipe. The potential adverse effects of the stormwater on groundwater quality are an important consideration (Livingston et al. 1988; Urbonas and Roesner eds. 1986). The soils should be moderately (in non-seismic areas) to well (in seismic areas) consolidated open graded sands or gravels, ground surface slopes should not exceed about 5% (with pipe invert slopes typically no greater than 0.5–1.0%), and there should be no expansive, collapsible or dispersive soils present (University of Wisconsin 1990).

Deliberate exfiltration provides underground disposal of stormwater runoff in situations where its accumulation on the ground surface would be undesirable, frequently in locations where there is no defined surface

drainageway available for drainage outfall. To accomplish this, perforated pipe is used, or the storm sewer is installed with open (cracked and unsealed) joints. A grit chamber (sediment trap), easily accessible for solids removal, should be installed downstream from stormwater inlets to prevent accumulation of settleable solids in the storage/exfiltration pipe.

The permeability of soils into which effluent will be exfiltrated should be determined prior to design. Since some fine street washings inevitably will pass the grit chamber, and may tend to reduce natural permeability over the long-term, the design infiltration rate used for determining the size of necessary storage/exfiltration piping should be less than the soil's natural infiltration rate. Location of a grit chamber downstream from storm inlets, rather than usage of combined inlets/catch basins, will increase the "catch" of suspended solids by deposition at locations free from the turbulence usually present in an inlet box.

The balance between in-pipe storage capacity and accumulation of runoff on the ground surface (ponding) is usually a balance between cost and acceptable inconvenience or risk, a determination normally made by the client, project owner, or regulatory authority.

Precautions noted above for deliberate infiltration are equally applicable to exfiltration. Deliberate exfiltration should not be considered unless stormwater runoff quality is consistent with long-term maintenance of groundwater quality (an NPDES permit may be necessary).

B. Inadvertent Infiltration/Exfiltration

Inadvertent or unanticipated infiltration occurs when storm drains lose their structural integrity, when tree or shrub roots penetrate a storm drain and cause blockage, or when natural trench materials are piped into storm drains creating cavities that may collapse with subsidence of the overburden and thereby damage the pipe. Where stormwater runoff must be treated to improve its quality prior to its discharge (especially in some combined sewer systems), infiltration can increase the amounts of water which must be processed to unacceptable levels. Deliberate (often surreptitious) introduction of process or other wastewaters into a separate storm sewer system also may be viewed as unanticipated infiltration, but its control is an administrative problem.

Inadvertent infiltration usually can be prevented or minimized. Prevention entails use of:

(a) Non-brittle pipe materials to the extent possible.
(b) Sealed pipe joints that can absorb some movement without leakage.
(c) Proper bedding of pipes to minimize their potential movements.
(d) Backfilling of trenches with compacted soils materials that will limit percolation of surface waters into trenches.
(e) Avoidance of stones or other materials, in trench backfills, that might result in point loads on pipes.
(f) Careful supervision of construction to assure watertightness of com-

pleted storm sewer systems, possibly with post-construction system pressure testing.

(g) Attention to these factors throughout the entire drainage system, from extreme headwater appurtenances to points of storm sewer system discharge. Enforced ordinances may be desirable to prevent unauthorized discharges into storm sewers.

Inadvertent exfiltration occurs when runoff waters leak from a storm sewer and cause undesirable soil movements or degradation of groundwater quality. Particular care should be exercised to avoid exfiltration (leakage) into expansive clay soils as the water may induce damaging volume increases (heave or swell), into collapsible soils such as cemented loess deposits, into dispersive soils subject to piping and resultant collapse, or into karst (limestone) formations subject to the formation of solution channels. Exfiltration from a pressure sewer (where the hydraulic gradient may be some distance above the sewer) may be unusually damaging if leakage under head erodes trench materials and causes storm sewer blowout. Precautions necessary to prevent unwanted exfiltration are the same as those listed above for infiltration.

VIII. STREET AND INTERSECTION DESIGN

The following provides an introduction to street and intersection design. More detailed guidance can be found in most large city drainage criteria manuals (several of which are listed in the references to this chapter), and in references published by AASHTO, the Federal Highway Administration, and many state highway departments.

Design criteria for the collection and disposal of runoff water on public streets is based on a reasonable frequency of traffic interference. Depending on the character of the street, certain traffic lanes may be fully inundated with a frequency not exceeding the minor system design return period. Good drainage design should provide direct traffic benefits, lower street maintenance costs, and protect street paving and the subgrade from unnecessary deterioration.

A. CLASSIFICATION OF STREETS

Street drainage practices are dependent upon the type of street use and construction (Urban Drainage and Flood Control District 1984).

(a) Local—A local street is a minor traffic carrier within a neighborhood characterized by one or two moving lanes and parking along curbs, with no through traffic moving from one neighborhood to another. Traffic control may be by use of stop or yield signs.

(b) Collector—A collector street collects and distributes traffic between arterial and local streets. There may be two to four moving traffic lanes and parking may be allowed adjacent to curbs. Traffic on collectors has right-of-way over traffic from adjacent local streets.

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(c) Arterial—An arterial street permits rapid and relatively unimpeded traffic movement. There may be four to six lanes of traffic, and parking adjacent to curbs may be prohibited. The arterial traffic normally has the right-of-way over collector streets. Construction of an arterial will often include a median strip with traffic channelization and signals at numerous intersections.
(d) Freeway—Freeways permit rapid and unimpeded movement of traffic through and around a city. Access is normally controlled by interchanges at major arterial streets. There may be eight or more traffic lanes, frequently separated by a median strip. Parking normally is not permitted on the freeway right-of-way.

B. Effect of Stormwater Runoff on Street Traffic Capacity

Storm runoff that influences the traffic-carrying capacity of a street can be classified as follows (see Figure 8.7) (Urban Drainage and Flood Control District 1984):

(a) Sheet flow across the pavement once rain strikes the pavement and flows to the edge of the street.
(b) Runoff flowing adjacent to the curb.
(c) Stormwater ponded at low points.
(d) Flow across the traffic lane from external sources or cross street flow, (as distinguished from rain falling on the pavement surface).
(e) Splashing on pedestrians.

C. Storm Drainage Design Criteria for Urban Streets

1. General Guidance

The gutter grade refers to the grade of the gutter parallel to the direction of flow. The maximum allowable grade for a gutter is not governed by drainage, but the allowable capacity of gutters on excessively steep slopes is limited. The minimum allowable gutter grade, to facilitate proper drainage should be 0.4%.

The maximum allowable crown slope is not affected by drainage requirements, but the minimum crown slope of the street should be 2% to facilitate drainage from the pavement.

The standard gutter, together with a curb, should be at least 6 inches deep and 2 feet wide, with the deepest portion adjacent to the curb. Other gutter configurations may be used as conditions require. Some cities use higher curbs to allow for maintaining runoff capacity after repaving, though there are no recognized design criteria for maintaining such capacity. The conservative designer may choose to allow for the fact that each repaving reduces capacity.

Driveway entrances should be recessed into curbs. The driveway should slope up to an elevation equal to the top of the curb so runoff within the street cannot flow onto adjacent property through the driveway entrance.

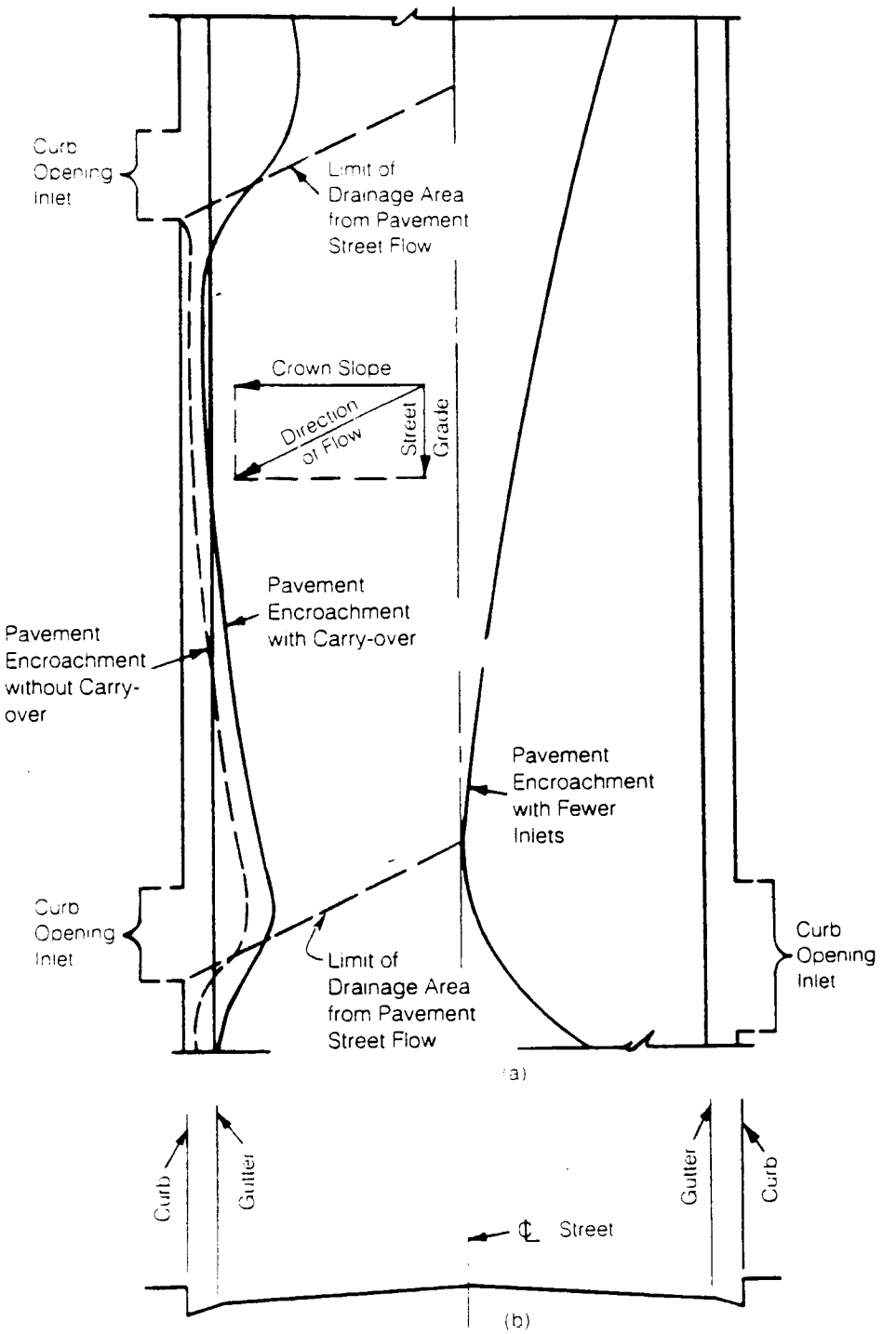


Figure 8.7—Diagram of gutter and pavement flow patterns: (a) plan view of street with inlets and (b) section (Urban Drainage and Flood Control District, 1984).

Inverted crown or dished streets should not be used, since during times of flooding they cannot function as a vehicular traffic carrier. Other disadvantages of dished streets include:

(a) Concentration of snowmelt runoff at center of street where it can then refreeze, causing a significant traffic hazard.
(b) Lawn irrigation and other low flows continually affect traffic.
(c) Any pavement cracking allowing leakage to the subbase affects the traffic-carrying portion of the street first.
(d) Difficulty in designing stormwater inlets.
(g) Lack of damage-resistant pavement designs and materials.
(h) Safety hazard during flooding.

2. *Cross Fall*

The term cross fall refers to the difference in elevation between the gutter flow lines on opposite sides of a street. Under most conditions streets are designed with zero cross fall. In hilly areas, and particularly at intersections in hilly areas, it may be necessary to construct the curbs at different elevations, resulting in cross fall.

Figure 8.8 (Urban Drainage and Flood Control District 1984) illustrates the loss of capacity of the higher gutter in a street with cross fall. When calculating allowable flow for the higher gutter, the actual flow prism configuration must be used. The capacity of the lower gutter may or may not be decreased, depending upon the street design. As with the upper gutter, the actual flow prism must be used in calculating allowable capacity.

When calculating the volumes of flow in each gutter, note that the upper gutter may fill quickly, by virtue of its location on the side of the street which will be receiving drainage from adjacent areas. Flow will then proceed across the crown of the street and into the opposite gutter. On local streets this is acceptable. However, on major streets, the interference to traffic movement due to the water flowing across the traffic lanes usually is unacceptable.

To prevent low flows from flowing across the traffic lanes, adequate capacity must be maintained in the higher gutter. To preserve this capacity the crown should be maintained within the one-quarter points of the street as shown by Section B-B of Figure 8.8.

On local streets, where cross fall is necessary due to the existing topography, inlets may be placed in the lower curb, and the street crown removed to allow flow from the upper curb to reach the inlet in the lower curb at specified locations.

3. *Street Capacity for Minor Storms*

Determination of street carrying capacity for the minor storm is based on two considerations; (1) pavement encroachment for computed theoretical flow conditions, and (2) an empirical reduction of the theoretical allowable rate of flow to account for practical field conditions.

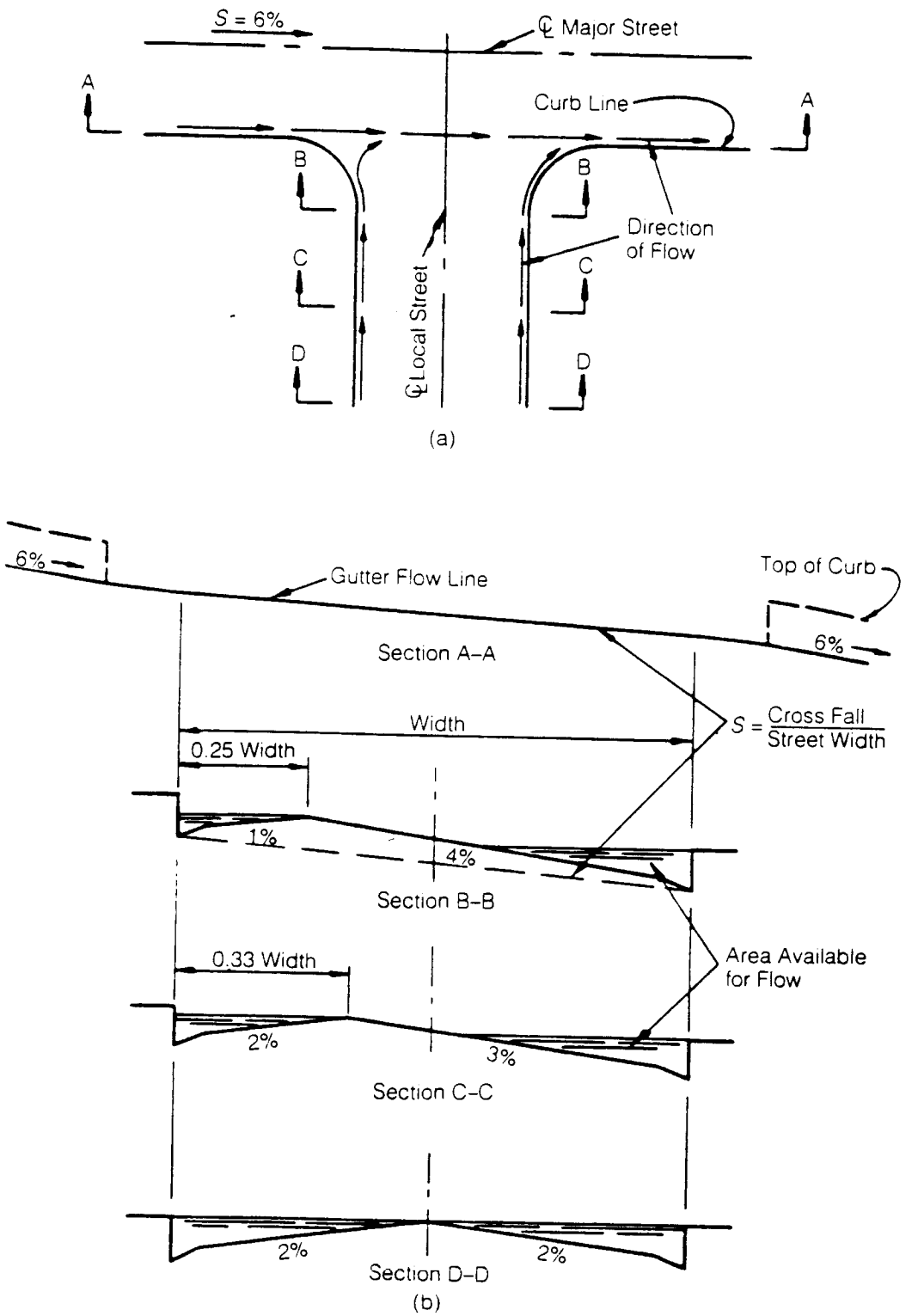


Figure 8.8—Typical intersection construction at junction of local and major street: (a) plan and (b) sections (Urban Drainage and Flood Control District, 1984).

Pavement Encroachment The pavement encroachment for the minor storm is often limited as shown in Table 8.3, although there is considerable variability from community to community, and local standards should be consulted. The storm sewer system should begin where the maximum encroachment is reached.

Calculating Theoretical Capacity When the allowable pavement encroachment has been determined, the theoretical gutter carrying capacity for a particular encroachment can be computed using the integrated form of the Manning equation, described in Chapter 6. To simplify computations, graphs for particular street shapes may be plotted, or computer programs can be used.

Allowable Gutter Flow The actual flow rate allowable per gutter can be calculated by multiplying the theoretical capacity by the corresponding factor obtained from Figure 8.9. The designer can then develop discharge curves for standard streets.

4. Street Capacity for the Major System Design Runoff

Determination of the allowable flow for the major system design runoff is based on allowable depth and inundated area, and the reduced allowable flow due to velocity considerations. In sump areas, overflow outlets (to parking or other graded areas) should be provided to prevent water in sumps, particularly when the sump is clogged, from entering adjoining buildings.

Allowable Depth and Inundated Area The allowable depth and inundated area for the major storm are normally limited per criteria such as in Table 8.4. The theoretical street carrying capacity then can be calculated using the Manning equation.

Ponding Ponding refers to areas where runoff is restricted to the street surface by sump inlets, street intersections, low points, intersections with drainage channels, or for other reasons. Limitations on pavement encroachment by ponding can be based upon recommended flood depths in Tables 8.3 and 8.4.

TABLE 8.3. Typical Allowable Pavement Encroachment for Minor Storm Runoff (Urban Drainage and Flood Control District 1984).

Local	No curb overtopping.* Flow may spread to crown of street.
Collector	No curb overtopping.* Flow spread must leave at least one lane free of water.
Arterial	No curb overtopping.* Flow spread must leave at least one lane free of water in each direction.
Freeway	No encroachment is allowed on any traffic lanes.

*Where no curb exists, encroachment onto adjacent property should not be permitted.

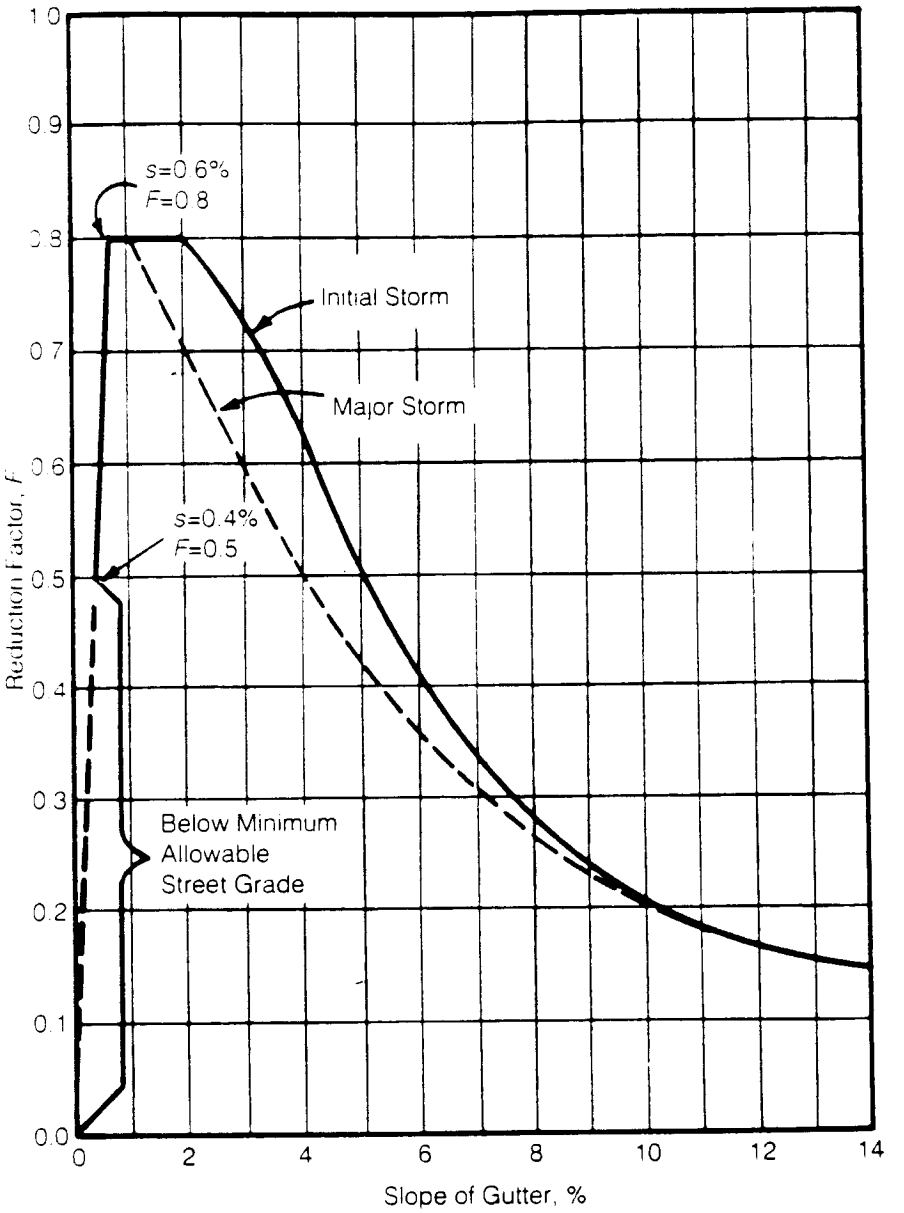


Figure 8.9—Reduction factor for allowable gutter capacity for local and collector streets (apply reduction factor for applicable slope to the theoretical gutter capacity to obtain allowable gutter capacity approaching arterial street) (Urban Drainage and Flood Control District, 1984).

5. *Cross-Street Flow*

There are two general categories of cross-street flow. The first is runoff that has been flowing in a gutter and then flows across the street to the opposite gutter or to an inlet. The second is flow from some external source, such as a drainageway, which will flow across the crown of a street when the conduit capacity beneath the street is exceeded. Cross-

TABLE 8.4. Typical Allowable Street Inundation Criteria for Major Storm Runoff (See Also Local Criteria) (Urban Drainage and Flood Control District 1984).

Street Classification (1)	Allowable Depth and Inundated Areas (2)
Local and collector	Residential dwellings, public, commercial, and industrial buildings shall not be inundated at the ground line, unless buildings are flood-proofed. The depth of water over the gutter flow line shall not exceed an amount specified by local regulation, often 12 inches.
Arterial and freeway	Residential dwellings, public, commercial, and industrial buildings shall not be inundated at the ground line, unless buildings are flood-proofed. Depth of water at the street crown shall not exceed 6 inches to allow operation of emergency vehicles. The depth of water over the gutter flow line shall not exceed a locally-prescribed amount.

street flow depth is often limited. Table 8.5 gives an example of such limits, and local standards should also be consulted (note that, in Table 8.5 no cross street flow is permitted for the minor design runoff).

6. *Intersections*

The various criteria presented herein for street inundation, ponding and cross street flow can be used in combination for intersection design in a procedure that ultimately determines the number, type, and size of inlets required.

TABLE 8.5. Typical Allowable Cross Street Flow Depths (Urban Drainage and Flood Control District 1984).

Street Classification (1)	Minor Design Runoff (2)	Major Design Runoff (3)
Local	6-inch depth at gutter or in cross pans	18 inches of depth above gutter flow line
Collector	Where cross pans are allowed, depth of flow shall not exceed 6 inches	18 inches of depth above gutter flow line
Arterial	None	6 inches or less over crown
Freeway	None	6 inches or less over crown

When local streets intersect major streets, the grade of the major street should not be interrupted, if possible. Figure 8.10 illustrates the typical street cross-sections necessary for such an intersection. The figure assumes that the major street grade is 6%, the maximum allowable crown slope is 4%, the minimum allowable crown slope is 1%, and the crown must be maintained within the one-quarter points of the street.

Storm Sewer System When a storm sewer will be located in an intersection, inlets should be placed and sized so that encroachment on the intersection is equivalent to that allowed on the street for the design runoff. Figure 8.11 illustrates typical inlet locations for various categories of intersections.

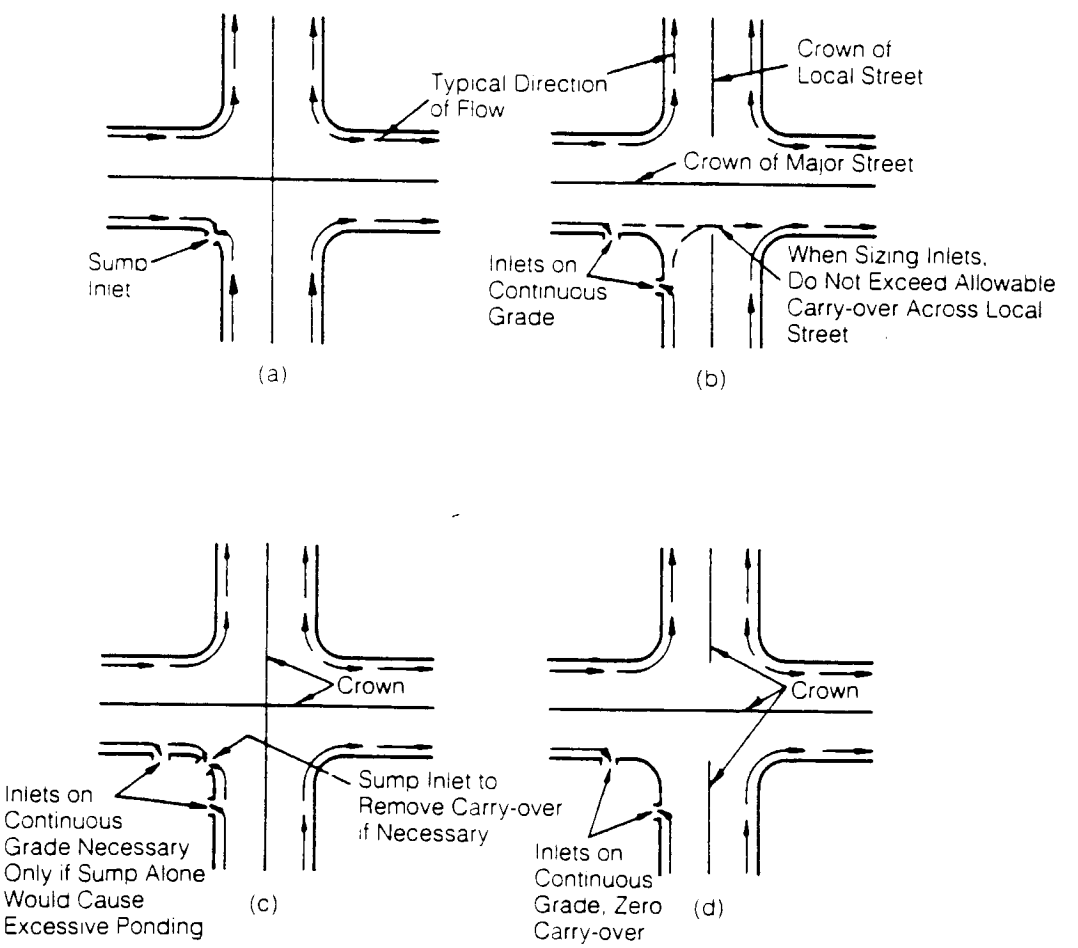


Figure 8.10—Typical street intersection drainage to storm sewer system. The examples show the minimum required inlets. Additional inlets may be necessary based upon allowable carrying capacity of gutters: (a) local street to local street, (b) local street to major street, (c) major street to major street—where crowns are to be maintained, and (d) major street to major street—one crown continuous (Urban Drainage and Flood Control District, 1984).

Drop Inlet Culverts When a storm sewer system is not required, and sufficient grade is available, a drop inlet culvert may be used to transport runoff beneath a street. Encroachment on the intersection should be limited to that allowed for the street.

Cross Drains Conventional cross drains (cross pans) may be used to transport runoff across local streets when a storm sewer system is not required. The cross pan size and slope should be sufficient to restrict encroachment to that allowed on the street. If absolutely necessary, pans may be used on collector streets.

Covered cross pans may be used where frequent low flows are anticipated, although they are difficult to maintain, and therefore should be avoided where possible. The covered pans should carry at least the low flows, and must allow larger (design runoff) flows to pass over the pan without exceeding depth of flow allowed on the street. No form of cross pan should be constructed across an arterial street.

7. Special Considerations

No specific limitations are set for sheet flow. Designers should be aware of its existence and effects and take precautions to limit its occurrence.

Where there is heavy pedestrian traffic, depth and area limitations may need modification. As an example, streets adjacent to schools, while considered local from a vehicle traffic standpoint, are arterials from a pedestrian standpoint, and should be designed accordingly. Designing for the pedestrian is at least as important as designing for vehicular traffic.

Where commercial buildings are constructed to property lines, the reduced clearance between buildings and heavy traffic must be considered. Splash from vehicles striking gutter flow may damage store fronts and make walking on sidewalks impossible. Ponded water and gutter flow exceeding 2 feet in width are difficult to negotiate by pedestrians (Urban Drainage and Flood Control District 1984).

As emphasized repeatedly in this manual, the criteria listed above apply principally to newly-developed areas. In established areas, it may be appropriate to accept the system as it exists, depending on the degree of hazard present, or to make some modifications (but not necessarily in accordance with the criteria).

D. Storm Drainage Design Criteria for Rural Streets

Rural areas (or areas with low-density development) have streets that may use roadside ditches for drainage purposes, as opposed to curbs and gutters. Most requirements set forth for typical urban streets apply equally to rural streets.

Determination of rural street carrying capacity for the minor and major storms is based upon the following considerations:

(a) Pavement encroachment allowed.
(b) Maximum allowable velocity to prevent scour.

Site specific standards apply for these factors. As a guide, the designer can use Tables 8.3, 8.4, and 8.5, and adjust the criteria to reflect rural conditions. Note, however, that future curb/gutter installation may reduce roadside runoff detention significantly, completely changing drainage responses. It is prudent to anticipate future requirements.

Drainage ditches adjacent to rural roads are common. As reach-by-reach evaluation of probable channel stability is not economically feasible, the guidance in Table 8.6 should be helpful. Note that velocities shown in Table 8.6 are maximum velocities. Design velocities should be no more than 75% of these values.

Design velocities for all linings should not fall below 2 feet per second for the minor storm runoff to minimize sediment deposition problems, unless lower velocities are required for water quality enhancement—in which case provision should be made to control erosion/sedimentation. The allowable capacity for the drainage ditch should be calculated using Manning's formula with an appropriate n value. If the natural channel slope would cause excessive velocity, drop structures, checks, riprap, or other suitable channel protection should be employed. Design depths are ideally limited to 1.5 feet, and preferably less than 1.0 foot.

IX. MAJOR DRAINAGEWAYS (OPEN CHANNELS)

Major drainage is the cornerstone of an urban storm runoff system, and will function whether or not it has been planned and designed. If a conveyance route for the major flood is not provided, flood waters

TABLE 8.6(a). Typical Permissible Velocities for Roadside Drainage Ditches with Erodible Linings (Site Specific Analysis Recommended) (Chow 1959).

Soil Type or Lining (earth, no vegetation) (1)	Permissible Velocity (fps) (2)
Fine sand (noncolloidal)	2.5
Sandy loam (noncolloidal)	2.5
Silt loam (noncolloidal)	3.0
Ordinary firm loam	3.5
Fine gravel	5.0
Stiff clay (very noncolloidal)	5.0
Graded, loam to cobbles (noncolloidal)	5.0
Graded, silt to cobbles (noncolloidal)	5.5
Alluvial silts (noncolloidal)	3.5
Alluvial silts (colloidal)	5.0
Coarse gravel (noncolloidal)	6.0
Cobbles and shingles	5.5
Shales and hard pans	6.0

TABLE 8.6(b). Typical Permissible Velocities for Roadside Drainage Ditches Lined with Uniform Stand of Grass Covers (Well-Maintained Grass) (Chow 1959).

Cover (1)	Slope Range (percent) (2)	Permissible Erosion Resistant Soils (3)	Velocity (fps) Easily Eroded Soils (4)
Bermuda grass			
Crested wheat grass	0–5	6.0	5.0
Buffalo grass	5–10	5.0	4.0
Kentucky bluegrass	over 10	4.0	3.0
Smooth brome			
Grass mixture	0–5	4.0	3.0
	5–10	3.0	2.5
Lespedeza sericea			
Weeping lovegrass			
Yellow bluestem			
Alfalfa	0–5	3.0	2.0
Crabgrass			
Common lespedeza			
Sudangrass			

will move downgradient on their own accord—often through yards, homes and businesses. Thus, major drainage systems must be given high priority when considering drainage improvements. The major system may include features such as natural and artificial channels, long underground conduit outfalls, streets, property line drainage easements, and other water-carrying routes.

In small urbanized basins (typically less than 20–50 acres, depending upon topography, imperviousness, rainfall and other factors), it may be feasible to contain the major flow within storm sewers and/or streets. At some point in the system, however, major flows eventually exceed the combined capacity of storm sewers and streets, and a major drainageway is necessary.

The major system and minor drainage systems should be planned concurrently. A good major system can reduce or eliminate the cost of an underground storm sewer system, whereas an ill-conceived major system can make a storm sewer system very costly. The 2- or 10-year runoff can flow in the major system, but only a small portion of the major system design runoff will flow in the minor system.

The planner and engineer have great opportunities when working on a major drainageway to provide a better urban environment for all citizens. Benefits, in addition to flood control that often can be achieved, include creation of wetlands and other kinds of wildlife habitat, parks, trails and other recreation areas, and groundwater recharge zones. Properly designed, installed, and maintained channels can also reduce pollutant loads and mitigate sediment problems. The following discus-

sion of open channels is, of course, not limited to the major system, and many of the concepts apply to the minor system as well.

Open channels have significant advantages with regard to cost, capacity, multiple use for recreational, aesthetic, and other purposes, and potential for providing transient storage. Disadvantages include right-of-way needs and maintenance costs. The ideal channel is an old (geologically) natural one. The benefits of such a channel are that:

(a) Velocities are usually low, due to relatively flat longitudinal slopes, resulting in longer concentration times and lower downstream peak flows.
(b) Channel storage tends to decrease peak flows.
(c) Maintenance needs are usually low because the channel is somewhat stabilized.
(d) The channel provides social benefits (greenbelt, recreational benefits).

The more an artificial channel can be made to resemble a natural channel, generally the better it will be. It must be recognized however, that few natural channels would respond favorably to the hydrologic impacts of urbanization without man-made improvements, with channel stability leading the list of concerns.

In many areas about to be urbanized, the runoff has been so minimal that natural channels do not exist. However, thalwegs nearly always exist that provide an excellent basis for location and construction of channels. Good land planning should reflect these thalwegs and natural channels to reduce development costs and minimize drainage problems. In some cases, the wise use of natural water routes in the development of a major drainage system will obviate the need for an underground storm sewer system.

Other considerations with respect to open channel evaluation and selection include:

(a) Preservation/destruction of wetlands.
(b) Improvements in water quality.
(c) Trails/public recreation.
(d) Impacts on aquatic life.
(e) Wildlife habitat improvement or removal.
(f) Protection of views or open space.
(g) Historic/cultural preservation.

A. Choice of Channel

The choices of channel available to the designer are almost infinite. However, from a practical standpoint, the basic choice to be made initially is whether or not the channel is to be lined—structurally to accommodate higher velocities, or grassed to accommodate intermediate velocities. A variation of grass-lined channels is to simply add stabilization measures to natural channels, if increases in peak discharge are moderate. Concurrent with this evaluation is consideration of chan-

nel slope modification (cutting or drop construction) to reduce velocities. The choice must be based on a variety of factors which include:

(a) Regulatory—Federal, state and local regulations (see also Chapter 2).
(b) Hydraulic—Slope of thalweg, right-of-way, capacity needed, basin sediment yield, topography, ability to drain adjacent lands.
(b) Structural—Costs, availability of material, areas for wasting excess excavated materials.
(c) Environmental—Neighborhood character and aesthetic requirements, need for new green areas, street and traffic patterns, municipal or county policies, wildlife, water quality.
(d) Sociological—Neighborhood social patterns and child population, pedestrian traffic, recreational needs.

Prior to choosing the channel type the designer should be sure to consult with experts in related fields to assure that the channel chosen will create the greatest overall benefits. Whenever practical, the channel should have slow flow characteristics, be wide and shallow, and be natural in its appearance, and functioning (Urban Drainage and Flood Control District 1984; Livingston et al. 1988; Urbonas and Roesner 1986).

For open channel design, the general approach is to prepare profiles of routes that appear satisfactory and make rough cost estimates of each, using approximations of the character and location of channels or conduits. Include costs of bridges, drops, special structures, utility relocations, land acquisition, and other such factors. The advantages and disadvantages of potential routes are then examined with an environmental design team, which may include an urban planner, an attorney knowledgeable in drainage law, a landscape architect, and an urban sociologist.

The reader is referred to Chapters 9, 13, and 14 for detailed discussion on special structures commonly found in channels (riprap, energy dissipators, drop structures, etc.), channel linings, scour, structural design, and construction.

B. Hydraulic Analysis

The following sorts of information will be needed in the design of open channels, (discussed in detail in Chapter 6) (adapted from University of Colorado 1989 with modifications):

1. *Channel Geometry*

(a) The cross-section.
- Perpendicular to the direction of flow.
- Encoded as station and elevation pairs.
- Related to each other by distance between sections.

- Forms a three-dimensional digital model of floodplain and channel.

(b) Guidelines for cross-section spacing.
- Cross-sections should be located where there is an appreciable change in cross-sectional area, roughness, or gradient.
- Detailed cross-sections should be located above, below, and within bridges to define transitions.
- Cross-sections are needed at all control sections.
- Cross-sections are required immediately above and below a confluence on a main stream and above the confluence on the tributary.
- In general, more cross-sections are needed to define energy losses in urban areas as opposed to rural, where steeper slopes are encountered, and on smaller streams.
- Based on computed results, additional cross sections may be required to provide accurate results. The reach is too long if the energy slope decreases by more than 50% or increases by more than 100%, and flow distribution should be reasonable from section to section.

(c) Sources of topographic data.
- Aerial photography/photogrammetry.
- Field surveys.

2. *Hydraulic Roughness Values and Loss Coefficients*

(a) Sources of roughness values.
- U.S. Geological Survey Water Supply Papers 1849 and 1898-B (See references for other studies).
- Tabulation of *n*-values in textbooks (French 1985; Chow 1959).
- Photographs of channels and floodplains with calibrated *n*-values are useful to compare with field conditions.
- Formulas for computing roughness—these require additional data and may not account for all roughness factors.
- Can be calculated using HEC-2 if measured water surface profile elevations are available.

(b) Use of "n" values.
- May reflect more than just friction loss, including the effect of bed forms, vegetation and debris, and urbanization in the floodplain.
- Complex roughness conditions—can assign "n" values to streets, lawns and sidewalks and deduct the space occupied by buildings, or use composite values and manipulate the "n" values to account for blockage (for example, if 50% blockage by buildings then *n*-values increase by factor of 2).
- Design "n" should reflect minimal maintenance.

(c) Contraction and expansion losses.
- Referred to as "minor losses" and are applied to the change in velocity head.
- 0.1 to 0.3 for contractions.
- 0.3 to 0.5 for expansions.
- Are not significant if velocity changes are small.
- Are significant at encroachments where large velocity head changes occur such as at bridges.

3. *Stage/Discharge Data*

(a) Use of control sections for determining stage/discharge relationships.
(b) Controls due to critical depth at changes in thalweg profile from mild to steep, at weirs and other hydraulic drops, and at severe contractions in thalweg section.
(c) Controls due to uniform flow—artificial prismatic channels (or natural channels that have a constant cross-section) with a uniform grade.

4. *Data Sensitivity*

(a) Cross-section data.
- Scale of mapping—larger scale mapping (USGS quads) can differ substantially from higher resolution mapping.
- Spacing also has a direct impact on computed results.

(b) Roughness data.
- Probably the most important parameter since conveyance is inversely proportional to the "n" value.
- "n" values should be selected to be conservative for the objectives of the project—upwardly conservative values should be used for flood studies, and downwardly conservative for scour and erosion analysis.
- Consistent with **observed** past maintenance performance.

(c) Stage/discharge data.
- Error in the estimated value for initial water surface elevation rapidly decreases as the calculation progresses.
- Errors in discharge estimation can be significant. If uncertainty exists then a sensitivity analysis should be conducted, using confidence bands for probability distribution.
- Relatively large error in discharge may have only moderate effect on stage forecasts. Do not split hairs.

C. Concrete-Lined Channels

Where the project requires a lined channel because of hydraulic, topographic, or right-of-way needs, concrete lining is usually chosen, although soil cement and roller-compacted concrete are gaining acceptance (see also Chapters 9 and 13). Whether the flow will be supercritical or subcritical, the lining must be designed to withstand the various forces and actions which tend to overtop the bank, deteriorate the lining, and erode unlined areas. Figure 8.11 provides a cross-section for a typical concrete channel. While concrete channels have high hydraulic capacities, they are usually the most expensive to maintain over time, because of settlement, cracking, and weed growth in joints.

1. *Supercritical Flow*

Supercritical flow in an open channel in an urbanized area creates certain hazards which the designer must take into consideration (Williams 1990; Urban Drainage and Flood Control District 1984), and is

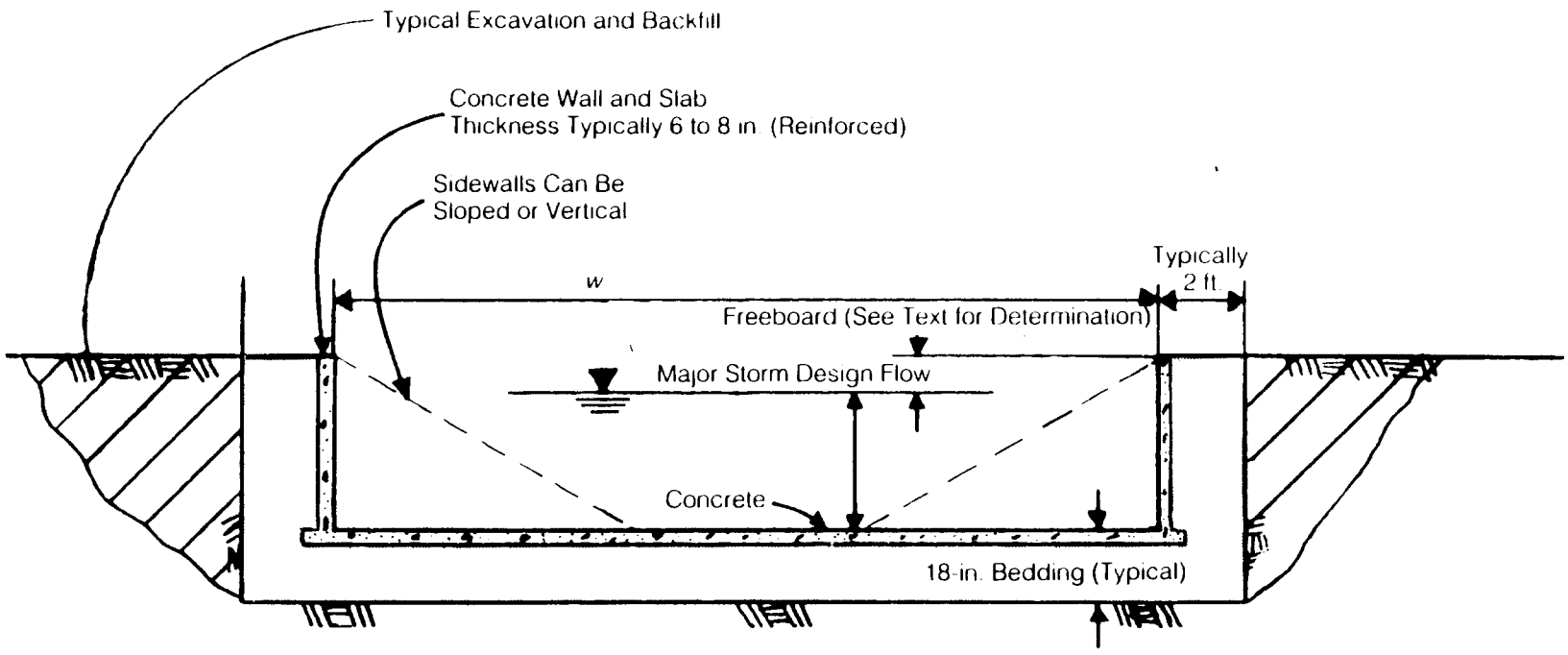


Figure 8.11—Schematic of concrete-lined channels (typical details not shown) (in. × 25.4 = mm and ft × 0.304 8 = m).

best avoided altogether. From a practical standpoint it is generally not advisable to have any curvature in such a channel. Careful attention must be taken to insure against excessive oscillatory waves which may extend down the entire length of the channel from only minor obstructions upstream. Imperfections at joints may cause their rapid deterioration, which in turn may cause a complete failure of the channel.

All channels carrying supercritical flow should be lined with concrete, which is continuously reinforced both longitudinally and laterally (San Diego County 1985). There should be no diminution of wetted area cross-section at bridges or culverts. Freeboard should be adequate to provide a suitable safety margin (a calculation procedure is provided in the next section). Bridges or other structures crossing the channel must be anchored to withstand the full dynamic load that might be imposed on the structure in the event of major trash plugging.

The concrete lining must be protected from hydrostatic uplift forces, which are often created by a high water table or momentary inflow behind the lining from localized flooding. Generally, a perforated, free-draining underdrain pipe will be required under the lining. With supercritical flows, minor downstream obstructions do not create backwater effects. Backwater computation methods are applicable for computing the water surface profile or the energy gradient in channels having a supercritical flow, however, the computations must proceed in a downstream direction. The designer must insure against the possibility of unanticipated hydraulic jumps forming in the channel. Flow at Froude numbers near 1 is unstable and should be avoided.

Roughness coefficients for lined channels are particularly important when dealing with supercritical flow. Rough channels may aggravate oscillatory wave tendencies. Once a particular roughness coefficient is chosen, construction inspection must be carried out in a manner to insure that the particular roughness is obtained. Because of field construction limitations, the designer should not use a Manning's "n" lower than 0.013 for a well-trowelled concrete finish. Other finishes should have proportionately larger "n" values assigned to them. Problems have arisen in the past (Williams 1990) when high bed loads have increased the effective "n" value to the point that flow shifts from supercritical to subcritical, leading to over-bank flooding. The designer may wish to adopt considerably larger "n" values as a safety margin to account for sediment and debris. For example, some individuals advocate "n" values of 0.04 to 0.05 for concrete channels specifically for this reason.

2. *Subcritical Flow*

Where subcritical flow is anticipated, wide, naturally-vegetated channels are normally preferable, if available thalweg slopes are steep enough, as channels of this nature are safer, provide open space and recreational opportunities, and may be more harmonious with natural geomor-

phologic processes (NAHB 1991; Urban Land Institute 1975). Concrete-lined channels may be needed in unusual circumstances (such as where right-of-way limitations exist).

Where the available slope is so small as to require that a smooth lining be used, the designer may find that desired roughness coefficients can be obtained by lining only the channel bottom. Such a practice should be confined to sites with stable soils.

The following recommended criteria for the design and construction of concrete-lined channels are typical recommendations that should be evaluated for applicability based on site-specific conditions.

Hydraulics

(a) Freeboard—Adequate channel freeboard above the designed water surface should be provided. One suggested method of calculating minimum freeboard is (Chow 1959):

$$H_{FB} = \sqrt{Cd} \tag{8-1}$$

where

H_{FB} = freeboard (feet).
C = a coefficient varying from 1.5 (for channel capacities of 20 cfs) to 2.5 (for channel capacities of 3000 cfs or more).
d = depth (feet).

Minimum freeboard is generally 1 foot, and additional freeboard must be provided to accommodate superelevation, standing waves, and/or other water surface disturbances. Freeboard usually should not be obtained by the construction of levees.

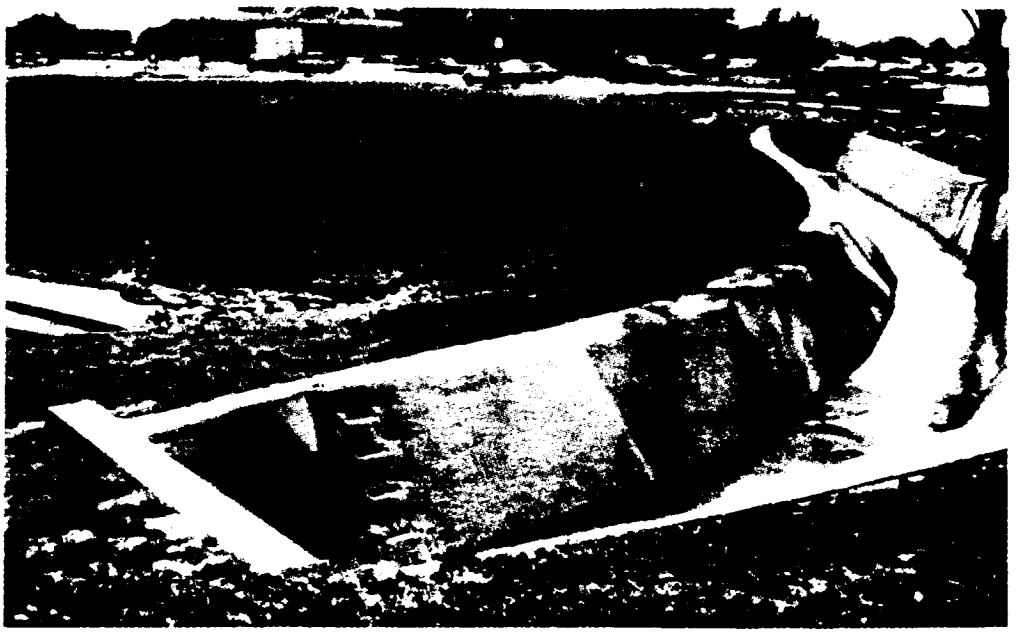
(b) Superelevation—Superelevation of the water surface must be determined at horizontal curves and design of the channel section adjusted accordingly.

An approximation of the superelevation can be obtained from the following equation:

$$h = \frac{V^2 Tw}{g r_c} \tag{8-2}$$

where

h = required superelevation (ft.).
V = velocity (feet per second).
r_c = centerline radius of curvature (feet).
Tw = top width of channel (feet).
g = gravitational constant (32.2 ft./sec).



Curved, concrete-lined channel. Note the overflow on the left side of the channel into the dry pond below.

(c) Velocities—Flow velocities should not exceed 7 fps or result in a Froude number greater than 0.8 during the 100-year flood for non-reinforced linings, and usually should not exceed 18 fps for reinforced linings (Urban Drainage and Flood Control District 1984).

Discussion regarding concrete materials, concrete lining sections, joints, finish, curing, and steel reinforcement is provided in Chapter 13. Earthwork, bedding, underdrains, and other structural and geotechnical design considerations are presented in Chapter 14.

D. Grass-Lined Artificial Channels

Grass-lined channels are desirable in many respects (Urban Land Institute 1975; Williams 1990). Channel storage, low velocities, and ecological (water quality enhancement) and aesthetic benefits create significant advantages over other types, although they generally require more land area. Their design must give full consideration to sediment deposition and scour, as well as hydraulics. Figures 8.12, 8.13, and 8.14 provide typical grass-lined channel cross-sections. Figure 8.15 provides a cross-section of a roadside ditch, which is a variation of a grass-lined channel.

1. Channel Stability/Protection

Allowable velocities, compatible with various types of soils and ground cover, typically published in handbooks, should be recognized as **maximum** velocities. Such information sources usually are silent with regard

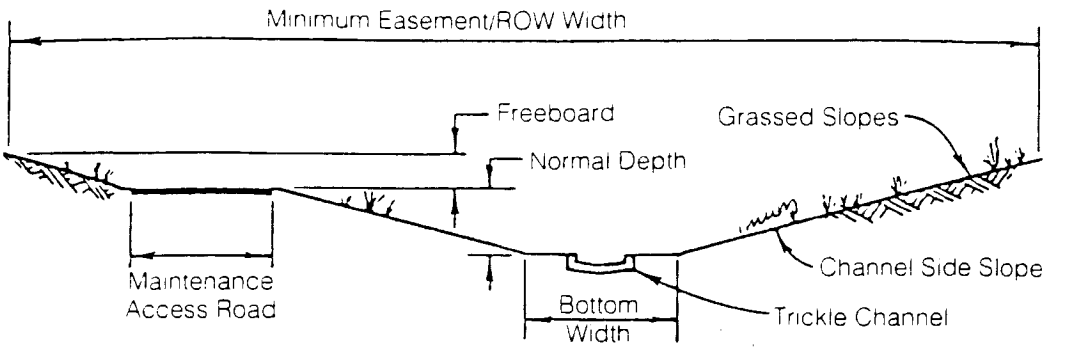


Figure 8.12—Typical grass-lined channel section type A: (1) bottom width—consistent with maximum allowable depth and velocity requirements, should not be less than trickle channel width; (2) trickle channel—minimum capacity normally specified locally—channel to be constructed of concrete, grouted riprap, or other approved materials; (3) normal depth—normal depth at design flow should not exceed a locally specified maximum value, maximum flow velocity at normal depth should not exceed a locally fixed value, frequently 7 fps (2m/s); (4) freeboard to be provided; (5) maintenance access road—minimum width typically 12 ft (4m); (6) easement/ROW width—maximum width to include freeboard and maintenance access road; (7) channel side slope—maximum side slope for grassed channels based on local criteria (3H:IV or 4H:IV typical); and (8) Froude number—maximum value typically must be met.

to the occurrence frequency and flow velocity durations related to the published data. These considerations remain essentially unquantified. The engineer is advised to be conservative when selecting design flow velocities, unless there are adequate **observed** erosion data available on nearby installations.

In non-cohesive soils, especially in arid areas lacking significant stabilizing vegetation, side slopes of channels typically will undermine during significant discharges and collapse (slough, slide) into the channel, and then be moved downstream. Bank stabilization, often to depths below scour depths, is important in such situations. The steeper the thalweg gradient, the more important such stabilization becomes.

Channel thalweg gradients must often be controlled to limit potential flow velocities (USBR 1974; USDA 1962). This can be done by various means, such as check dams, with stilling basins or other energy-absorbing structures provided immediately downstream. Such facilities should be designed so that the maximum foreseeable discharges will not overtop them, nor result in their undermining and loss. Where "head-cutting" (serious, progressive upstream bottom degradation) may occur, it is necessary to provide facility protection or extend gradient controls downstream to a location below which degradation will not occur.

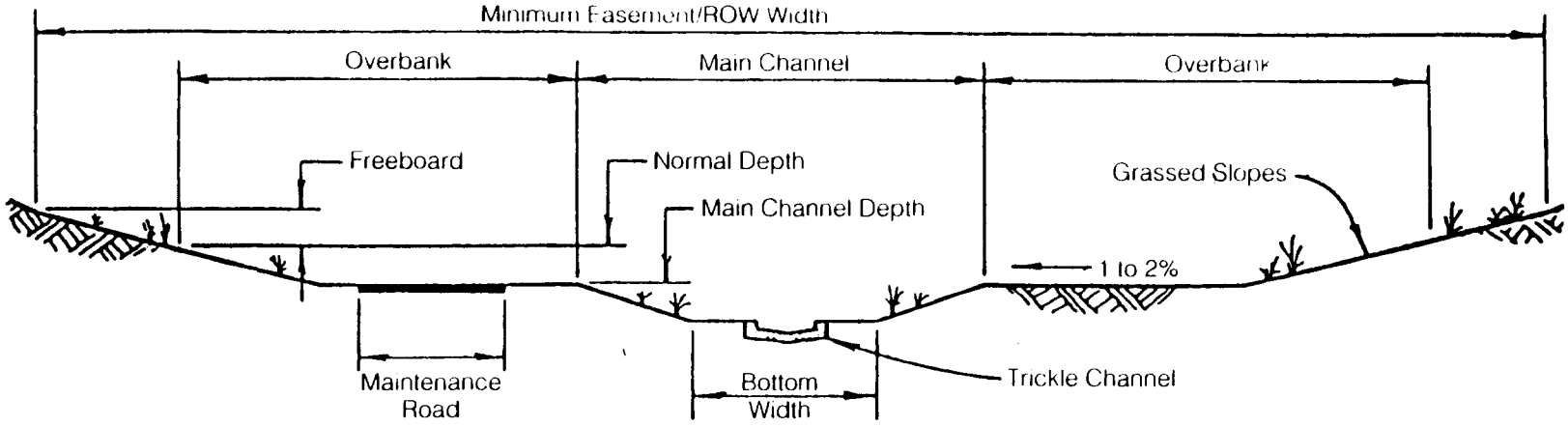


Figure 8.13—Typical grass-lined channel section type B: (1) main channel—typically a given percentage of design flow with maximum flow velocity specified; (2) trickle channel—minimum capacity normally specified locally; channel to be constructed of concrete, grouted riprap, or other approved materials; (3) normal depth—flow depth for design flow should not exceed a locally specified maximum value; (4) freeboard to be provided; (5) maintenance access road typically 12 ft (4m); (6) easement/ROW width—minimum width to include freeboard and maintenance access road; and (7) overbank—flow in excess of main channel to be carried in this area, area may be used for recreation purposes.

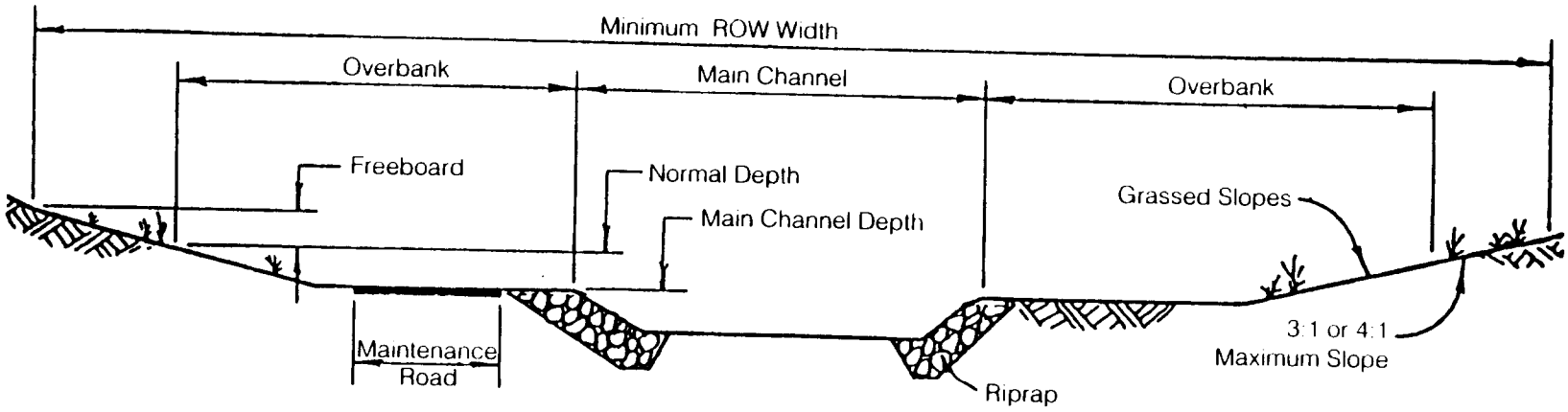


Figure 8.14—Typical grass-lined channel section type C: (1) this section may be appropriate for channels in sandy soils; (2) the main channel capacity is normally a specified percentage of the design flow, and it has a maximum allowable velocity; (3) easement/ROW width is minimum width to include freeboard and maintenance access road; (4) overbank—flow in excess of main channel to be carried in this area, area may be used for recreation purposes; (5) freeboard based on local criteria; (6) riprap sized for design flow in accordance with Chapter 9 procedures; (7) maintenance road width typically 12 ft (4m).

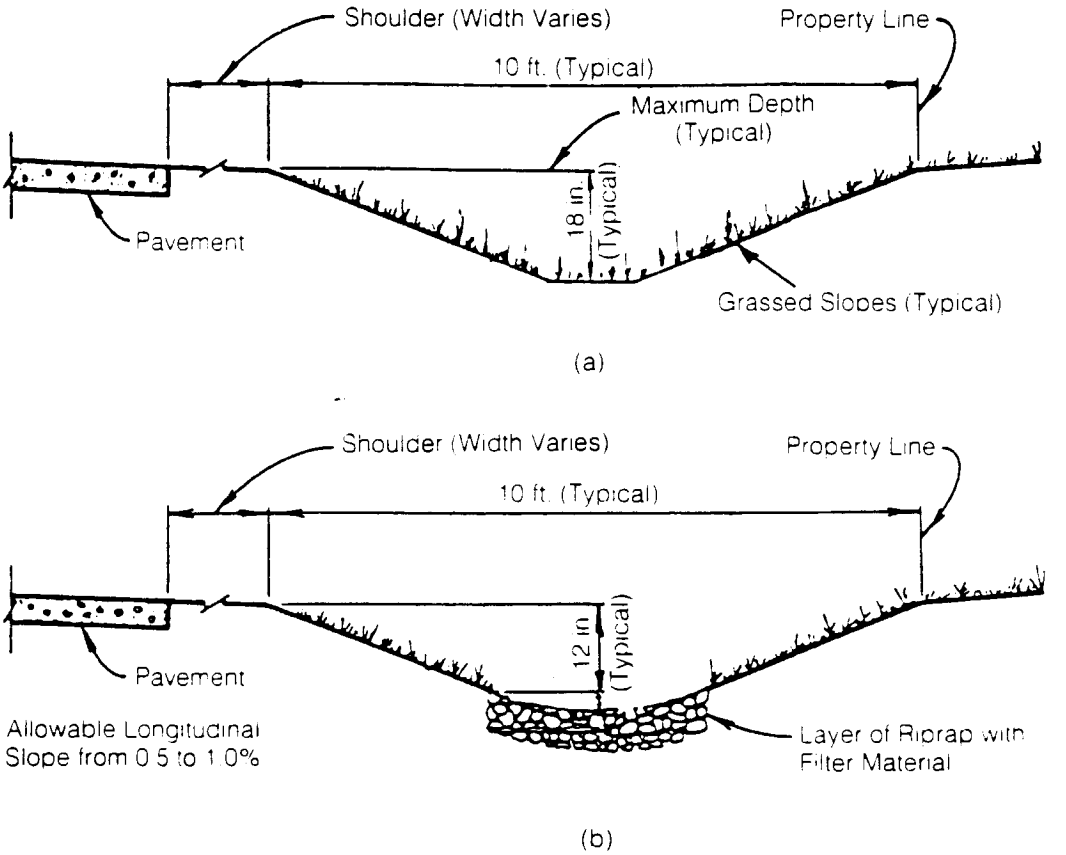


Figure 8.15—Typical roadside ditch sections (ft × 0.304 8 = m and in. × 25.4 = mm).

Generally, channels in non-cohesive soils in urban areas should have sufficient width to assure that their allowable velocities are not exceeded under maximum foreseeable discharges. This may result in unusual costs, unsightliness, or other undesirable effects, but is the price paid for continuing channel stability (U.S. Soil Conservation Service 1954, 1977).

Potential sediment loads always should be considered when planning any kind of ponded storage, or channel with a mild invert slope. Upstream erosion may have particularly adverse consequences where relatively small ponds are developed on-line with large upstream contributory areas. Development of ponds as off-line facilities, with inlets operated to fill the ponds when suspended solids loads in the waterway are minimal, is a wise approach when feasible.

There is a foreseeable relationship between stage-frequency characteristics and potential sediment loads for any given type of upstream soils (Urban Land Institute 1978). Where significant deposition occurs naturally, following mean-annual or smaller discharges, caution is advised. There is no practical way to overcome Stokes' Law. Bed loads will deposit when velocity diminishes. One of the most frequent deposition areas is upstream of culvert or bridge entrances, where signif-

icant headwater depth (with its associated velocity reduction) may be necessary to develop design discharge capacities.

2. *Preliminary Design Criteria*

The maximum velocity for the major storm design runoff should be low enough to keep scour problems within reasonable limits. Until a satisfactory grass cover is established, design flows will cause serious channel cutting and bank cutting at bends. Bends, transitions and the like merit careful evaluation, and often the designer will find it prudent to install erosion protection at key locations. More discussion of how to determine whether or not protection is required at channel discontinuities is provided in Chapter 9. Where the natural topography is steeper than desirable, drops should be used to keep velocities within desired limits. Drops are discussed in Chapter 9.

Channels will function better with longer radii of curvature. In general, the centerline radius of curvature should be at least twice the design flow top width, and not less than 100 feet.

Bridge deck bottoms and exposed pipe channel crossings often control the freeboard along urban channels. Where this is not the case, the allowance for freeboard should depend somewhat upon the conditions adjacent to the channel. For instance, localized overflow in certain areas may be desirable because of ponding benefits. In general, a minimum freeboard of 1 foot should be allowed (see discussion of freeboard on next page).

The grass species chosen must be sturdy, inundation and drought resistant, easy to establish, and able to spread after establishment. A thick root structure is necessary to control weed growth and erosion. The Soil Conservation Service and local landscape architects can provide assistance in selecting grass mixtures suitable for actual site conditions.

3. *Channel Cross-Sections*

Any channel shape suitable to the location and the environmental conditions may be used, so long as channel stability, public safety, and maintenance are not impaired. Often the channel configuration can be chosen to provide open space, recreational opportunities and wildlife habitat (Grove 1990). For example, in an effort to maximize open space, some communities have established 2–5 year capacity trapezoidal channels, and then formally designated adjacent overflow areas as regulatory floodplains (Urban Drainage and Flood Control District 1988, 1990). Varying channel characteristics from reach to reach is especially appealing to the public. Limitations within which design should normally fall for the major storm design flow include:

Side slopes The flatter the side slope, the better. Under special conditions, such as where development exists and right-of-way is a problem, the slopes may be as steep as 3:1 (5:1 is generally the safe limit for mowing equipment). Slope stability must of course be maintained, particularly during periods of rapid drawdown.

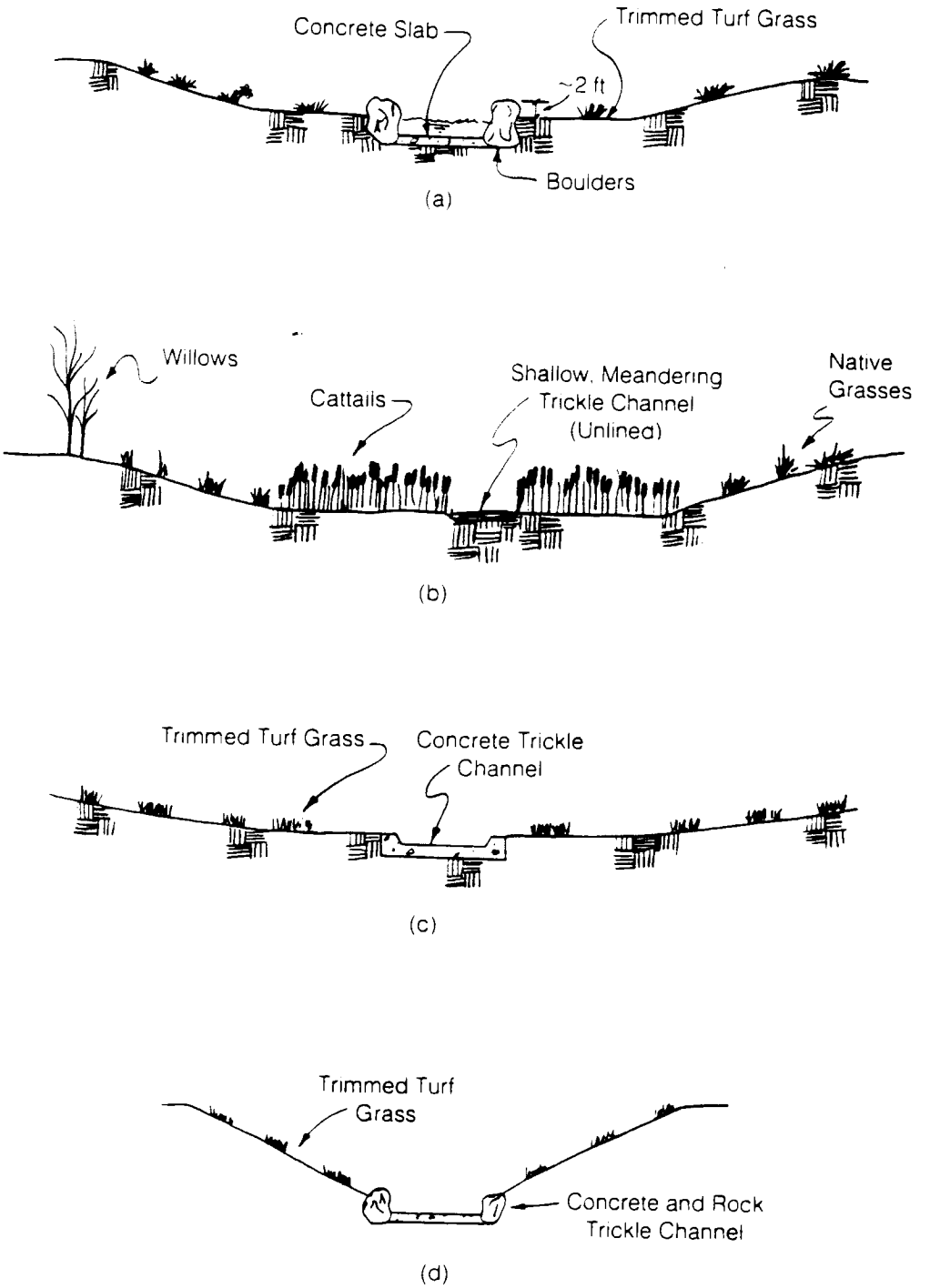
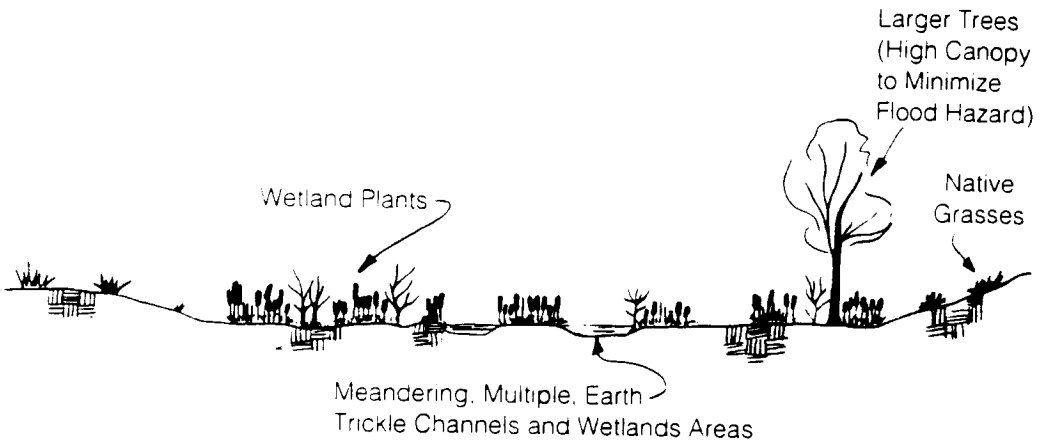
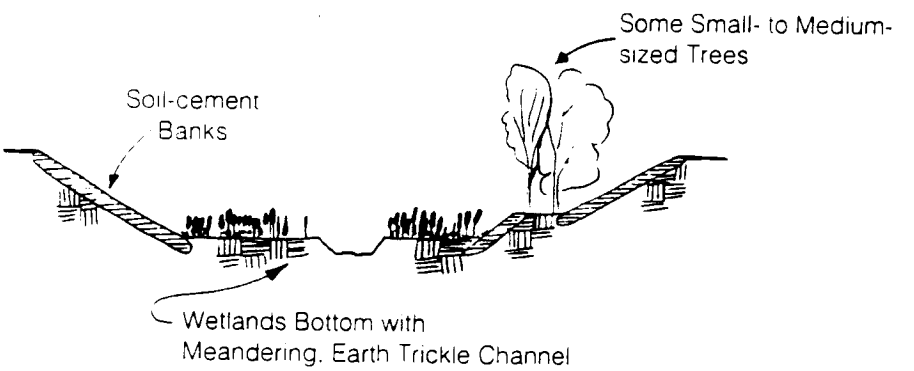


Figure 8.16a—Conceptual sketches for major drainageways with trickle channels: (a) dry bottom and rock-lined, concrete trickle channel; (b) wetland bottom and shallow, unimproved trickle channel; (c) dry bottom and shallow concrete "pan" as trickle channel; (d) steep sideslopes and rock-lined, concrete trickle channel. (Continued on next page)



(e)



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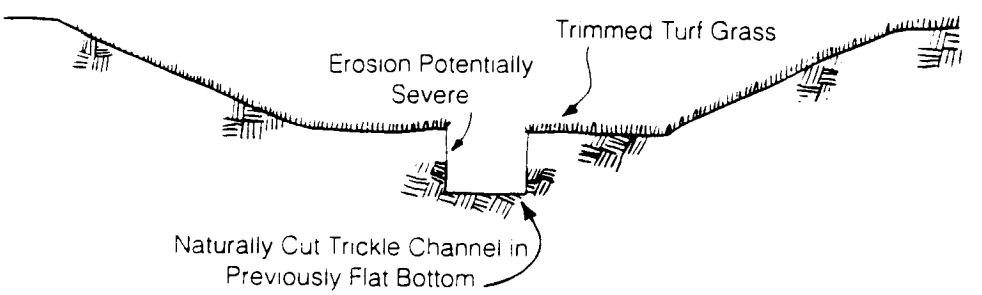

(g)

Figure 8.16b—(Continued from previous page) Conceptual sketches for major drainageways with trickle channels: (e) wetland bottom and multiple unimproved channels; (f) stabilized channel banks and meandering unimproved trickle channel; and (g) naturally developing unstabilized trickle channel.

Denver, Colorado—Grass-lined open channel with a rock-lined "trickle channel." Note the grouted-rock drop structures.

Bottom width The bottom width should be designed to accommodate the hydraulic capacity of the cross-section, recognizing the limitations on velocity and depth. Width must be adequate to allow necessary maintenance.

Trickle channels Trickle channels or underdrain pipes are sometimes used on urban grass channels to convey low flows (1–5% of design flow). In some circumstances, their use is being expanded to facilitate maintenance and to avoid creation of urban "wetlands." Under the current federal definition, nearly any location supporting water-loving vegetation can be classified as a "regulatory" wetland, subject to federal and state regulatory constraints (see also Chapter 2). In other situations, trickle channels are discouraged, so as to foster wetlands in channel bottoms (for water quality enhancement, groundwater recharge, wildlife habitat, etc.).

Concrete trickle channels are common because they facilitate maintenance and control base flow erosion. Other types are acceptable if properly designed. Trickle channels may not be practical on major streams and rivers or in large channels through sandy soils. Figure 8.16 provides typical trickle channel details, and some examples of how trickle channels can be incorporated into channel design.

4. *Freeboard*

Except where localized overflow (via side channel spillways or weirs) is desirable for additional ponding benefits or other reasons, freeboard should be provided. Freeboard requirements are normally specified locally; however, the following equation is generally applicable, at least for planning purposes.

$$H_{FB} = 0.5 + \frac{V^2}{2g} \tag{8-3a}$$

or, in metric units,

$$H_{FB} = 0.152 + \frac{V^2}{2g} \tag{8-3b}$$

The minimum freeboard should normally be at least 1 foot above the maximum design water surface elevation, although more is usually desirable to allow for a margin of error due to computational inaccuracies. An approximation of the superelevation can be obtained as indicated previously for concrete-lined channels.

5. *Curvature*

The centerline radius of curvature should be at least twice the top width of the channel, but not less than 100 feet.

6. *Right-of-way Width*

The minimum right-of-way width should include freeboard and adequate access for maintenance (including heavy equipment). Upstream from structures such as bridges or culverts, where floodwaters can leave the formal channel, additional right-of-way may be set aside to limit land development (Jones and Jones 1987).

7. *Roughness Coefficients*

The hydraulic roughness of man-made grass-lined channels depends on the length of cutting, if any, the type of grass, and the depth of flow, as well as the state of maintenance. Poorly maintained channels with large accumulations of sediment and debris have very high roughness coefficients. Typical roughness coefficients are given in Table 8.7. The 0.7 to 1.5 foot depth in Table 8.7 is generally suitable for computing the wetted channel portion for the minor storm runoff, while the greater than 3 foot depth is suitable for the major runoff computations. A depth of flow of 2.0 feet or more will usually lay the grass down to form a relatively smooth bottom surface (U.S. Department of Agriculture 1962).

Care must be exercised in operation and maintenance during periods following completion of construction, and before the grass stand has matured. While an "n" factor of 0.07 might be chosen for lower flows,

TABLE 8.7. Manning Roughness Coefficients, *n*, For Typical Grasses For Well-Maintained Straight Channels Without Shrubbery or Trees (Chow 1959).

Grass Type (1)	Depth of Flow 0.7–1.5 ft. (2)	Depth of Flow > 3.0 ft. (3)
Bermuda grass, buffalo grass, Kentucky bluegrass		
a. Mowed to 2 inches	0.035	0.030
b. Length 4–6 inches	0.040	0.030
Good stand any grass		
a. Length of 12 inches	0.070	0.035
b. Length of 24 inches	0.100	0.035
Fair stand any grass		
a. Length of 12 inches	0.060	0.035
b. Length of 24 inches	0.070	0.035

before the grass is up, the effective "n" may be as low as 0.025. Runoff during this period would have higher velocities and erosion damage might result.

8. Erosion Control

Practice has shown that it is uneconomical to design a grassed channel completely protected from erosion. It is preferable to provide a reasonable erosion-free design that includes allowances for additional erosion control measures and corrective steps after the first year of operation. However, the use of erosion control cutoff walls at regular intervals in a grassed channel is desirable (U.S. Department of Agriculture 1962). They will safeguard a channel from serious erosion in case of a large runoff prior to the grass developing a good root system and are also useful in containing the trickle channel.

Erosion control cutoff walls (see also Chapter 9) are often constructed of reinforced concrete, approximately 8 inches thick and from 1.5–3 feet deep, extending across the entire bottom of the channel. They can be shaped to fit a slightly sloped bottom to help direct water to the trickle channel or to an inlet. Depending upon scour, it may be necessary to anchor the wall into bedrock. Sloping rock structures are also common.

Grass will not grow under bridges, and the erosion tendency may be large. A cutoff wall at the downstream edge of a bridge is good practice, or the designer might choose to soil-cement the entire bottom width under the bridge deck.

At bends in the channel, special erosion control measures may be needed. However, once a good growth of grass is established and if the design velocities, depths, and curvatures are adhered to, erosion at bends will normally not be a problem.

To achieve the appropriate channel slope, the designer may find it necessary to use frequent drops. Erosion tends to occur at the edges and immediately upstream and downstream of drops, even though they may be only 6–18 inches high. Proper use of riprap at these drops is necessary. Remember that infrequent flows may override the drops, with resultant loss of the drop structure, unless deep cutoff walls and extensive toe erosion protection are provided.

9. *Water Surface Profile*

A water surface profile should be computed for all channels. Open channel flow in urban drainage is usually nonuniform because of bridge openings, curves, and structures (French 1985; Rouse 1950). Computation of the water surface profile should therefore use standard backwater methods, taking into consideration all losses due to changes in velocity, drops, bridge openings and other obstructions (see Chapter 6 for a discussion of these computations). Computations begin at a known point and extend in an upstream direction for subcritical flow. For this reason, hydraulic design of the channel should proceed in an upstream direction. The energy gradient should be shown on all preliminary drawings to help insure against errors. The water surface profiles and energy gradients may be shown on the final drawings (local practice will usually dictate the level of detail required for water surface profiles).

E. Natural and Composite Channels

If a natural channel is to be used for carrying storm runoff from an urbanized area, it may be assumed initially that the changed runoff regime will result in erosion (Urban Land Institute 1975). Careful hydraulic analyses must be made of natural channels to evaluate these tendencies. In many cases some modification of the channel will be required to create a more stabilized condition for the channel. Natural channels that include structural improvements are technically "composite" channels, although the distinction is subtle.

The investigations necessary to insure that a natural channel will be adequate are different for every waterway; however, the designer will generally find it necessary to prepare cross-sections of the channel for the major design runoff, to investigate the bed and bank material as to the particle size classification and to generally study the stability of the channel under future conditions of flow. Note that supercritical flow usually does not occur in natural channels (except in mountainous or other steeply sloping areas) and frequent checks should be made during the course of the backwater computations to insure that supercritical flow conditions are not present. Figure 8.17 provides a flowchart for the design procedures for high-gradient natural channels.

With many natural waterways it is necessary to construct drops and/or erosion cutoff check structures at regular intervals to decrease the thalweg slope and to control erosion. However, these channels should be left in as near a natural condition as possible, and extensive modi-

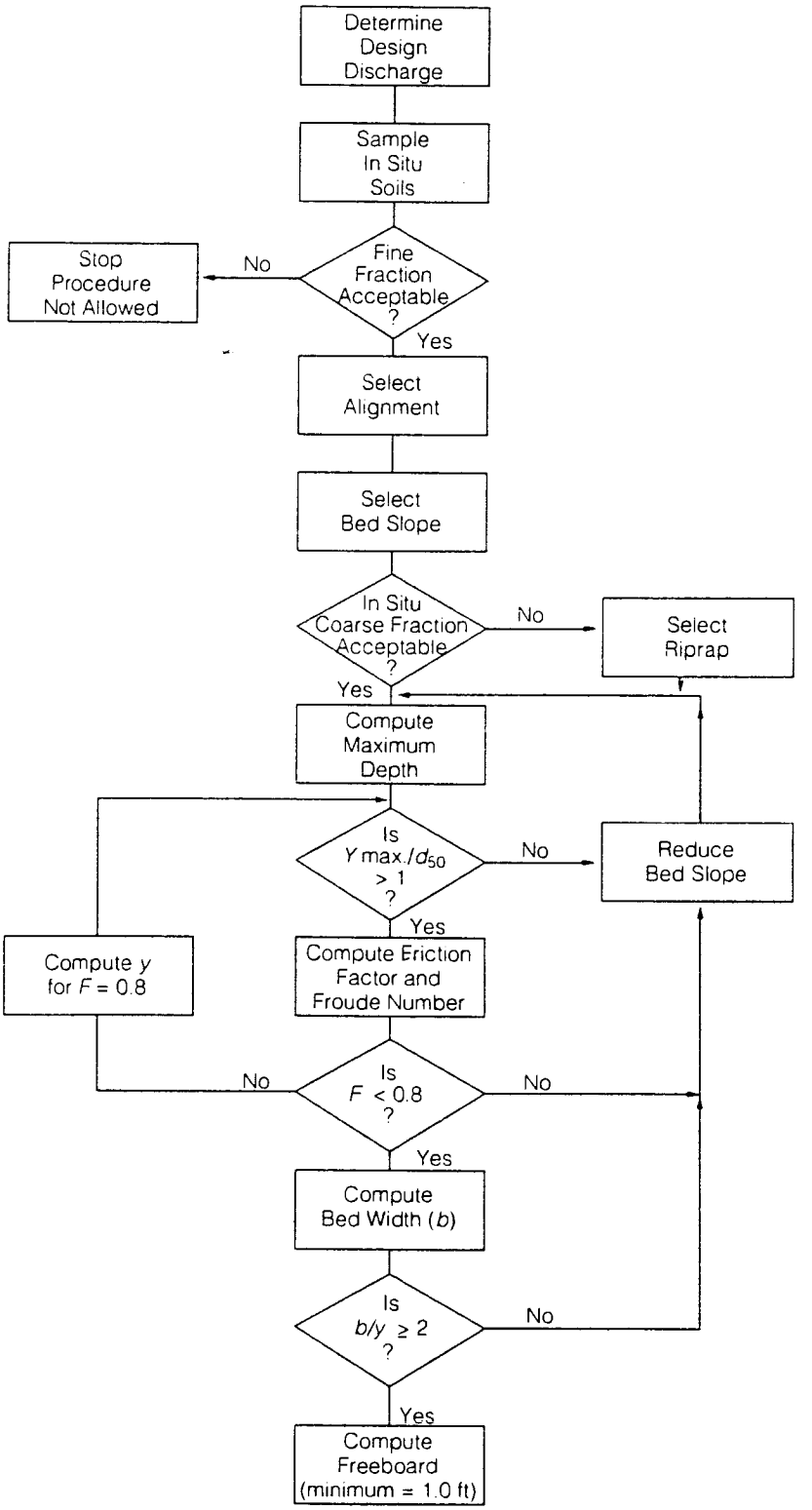

Figure 8.17—Design procedure schematic for high-gradient natural channels. (Boulder County, 1984).

fications should not be undertaken unless they are deemed necessary to avoid excessive erosion with subsequent deposition downstream (Note also that modification of the channel within the normal high water line may require a U.S. Army Corps of Engineers Section 404 permit).

The guidelines for design criteria and evaluation techniques for natural channels include:

(a) Channel and overbank areas should normally have adequate capacity for the major system design runoff (Urban Drainage and Flood Control District 1984). Freeboard is usually desirable.
(b) Natural channel segments with a Froude number greater than 0.80 for the major system design peak flows should be protected from erosion (Urban Drainage and Flood Control District 1984).
(c) Water surface profiles should be computed so that the floodplain can be zoned and protected (Federal Flood Insurance Administration 1981).
(d) Filling of the flood fringe reduces valuable channel storage capacity and tends to increase downstream runoff peaks. Filling of the flood fringe is usually subject to restrictions imposed by floodplain regulations (Federal Flood Insurance Administration 1981).
(e) Roughness factors (n) which are representative of **unmaintained** channel conditions should be used for the analysis of water surface profiles (Jones and Jones 1987).

Denver, Colorado—Small "pilot" channel that accommodates less than the 1-year runoff event with higher flows conveyed in the overbank area. Effective if the objective is maximizing the amount of open space to be set aside for the major drainageway.

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(f) Roughness factors (n) representative of maintained channel conditions should be used to determine **velocity** limitations.
(g) Plan and profile drawings of the floodplain should be prepared. Appropriate allowances for future bridges or culverts, which can raise the water surface profile and cause the floodplain to be extended, should be included in the analysis (Sheaffer et al. 1982).

The usual rules of freeboard, depth, curvature, and other guidelines applicable to artificial channels do not necessarily apply to natural channels. All structures constructed along the channel should be a minimum of 1 foot above the major system design water surface, or higher if the natural channel is in a mountainous area (subject to supercritical flow). Relative to maximizing open space, significant advantages may accrue if the designer incorporates into his planning the overtopping of the channel and localized flooding of adjacent areas, providing they are laid out and developed for the purpose of being inundated during the major storm runoff.

One variation of the natural channel is to leave the main channel area undisturbed (i.e., that area containing the base flow plus the immediate vegetation area) and to improve the overbank conveyance capabilities by excavating the floodplain area. This "naturalized" channel conveys the base flow and increases the capacity of the total channel to convey the major discharge (ASCE 1990). Figures 8.18 and 8.19 provide examples of slightly improved natural channels.

F. Other Channels

Other channels/materials discussed in Chapters 9 and 13, although not covered explicitly here, include:

(a) Rock/riprap channels.
(b) Soil cement/roller compacted concrete.
(c) Synthetic fabrics, such as woven fabric forms filled with concrete or grout, or discrete blocks on continuous backing.
(d) Special vegetation linings.
(e) Channel "rundown" (to convey minor flows from parking lots into outfalls). See Figure 8.20 for detail.

After the designer is thoroughly familiar with the applicability and limitations of the particular channel liner under consideration, and lacking specific local guidance for the subject material, he can adjust general criteria presented herein for concrete, grass, and natural channels to suit the particular application. For example, soil cement, roller-compacted concrete, synthetic fabrics and others would have general design parameters (freeboard, side slopes, maintenance requirements) **similar** to those for concrete channels.

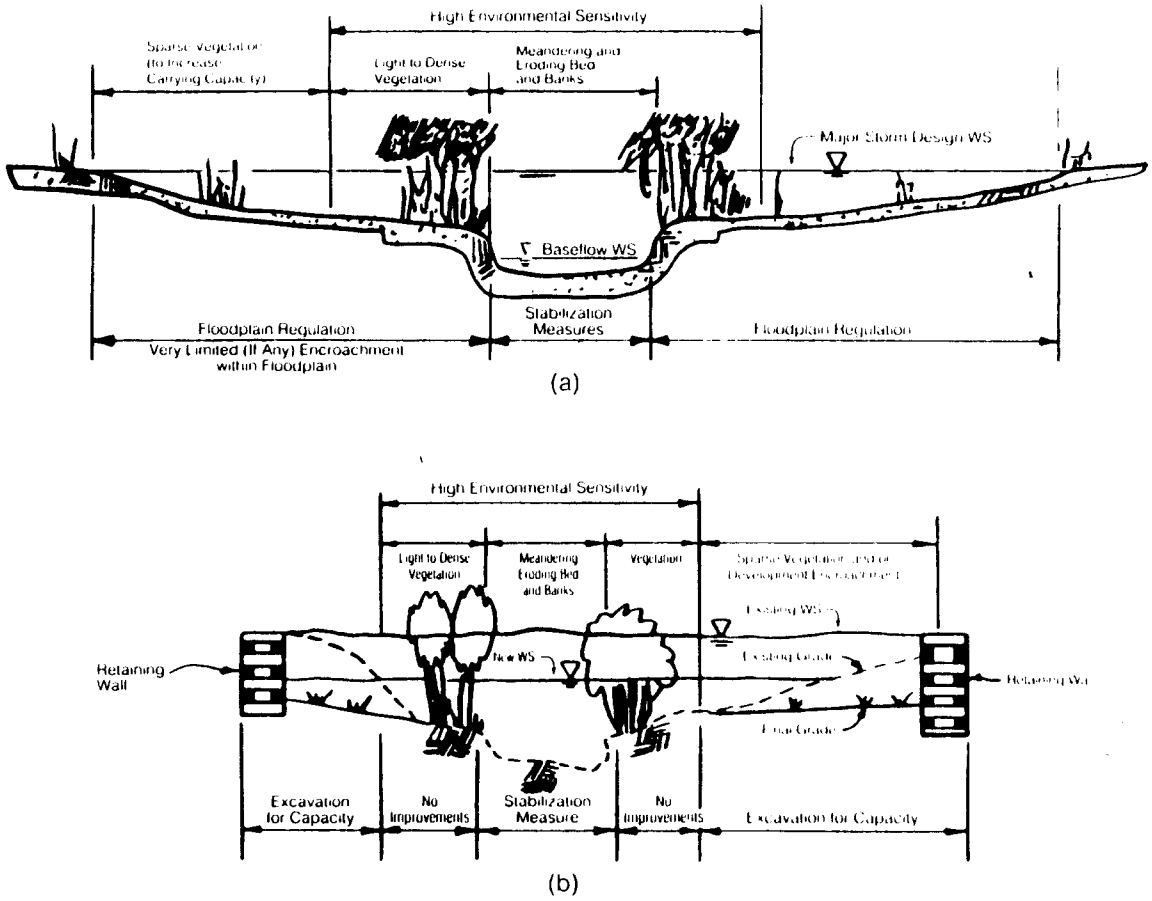

Figure 8.18—Typical natural channel: (a) stabilization measures may include check structures, riprap, minor grading, and short sections of retaining walls; generally little or no capacity improvements are included and (b) channel outside of environmentally sensitive area is excavated to increase capacity; improvements may require retaining walls; capacity improvements lower or confine design flood.

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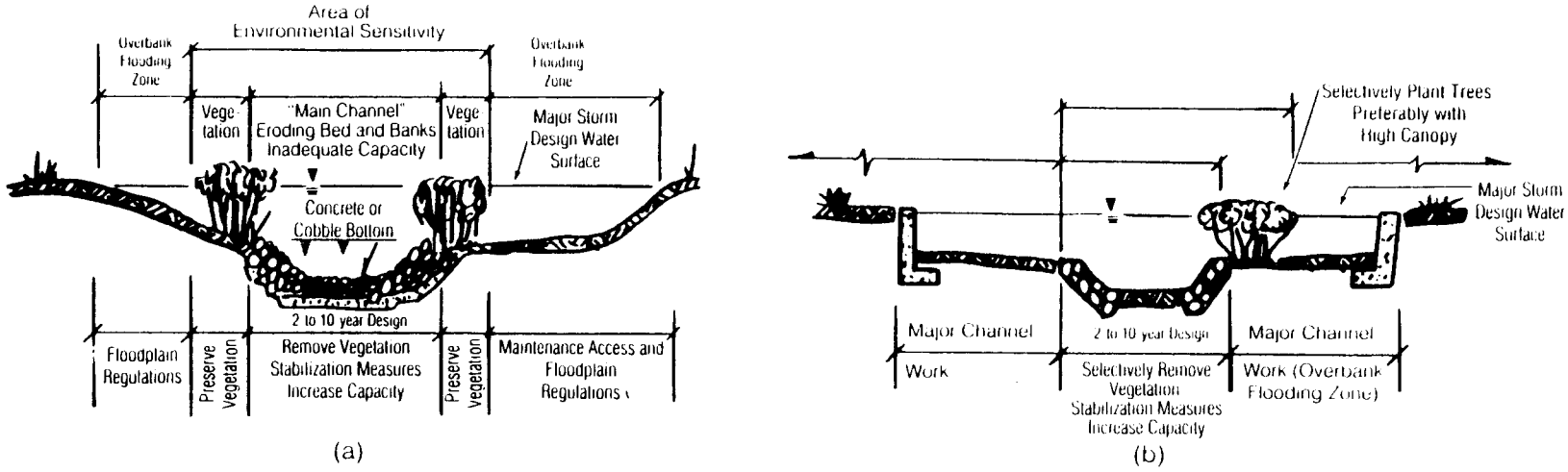

Figure 8.19—Storm drainage design considerations for composite channels: (a) stabilization measures may include check structures, riprap, grading, and retaining walls; main channel improvements increase capacity for 2- to 10-year floods and may reduce depth or confine major flood; (b) main channel improvements increase capacity for 2- to 10-year floods; complete channelization in outer floodplain area includes excavation, maintenance access, and retaining walls, improvements reduce and confine major flood; when stabilizing main channel and overbank, retain as much vegetation as feasible consistent with objectives of enhancing stability and capacity; encourage multiple uses of overbank flooding area, especially if main channel capacity is substantial (infrequent overbank flooding).

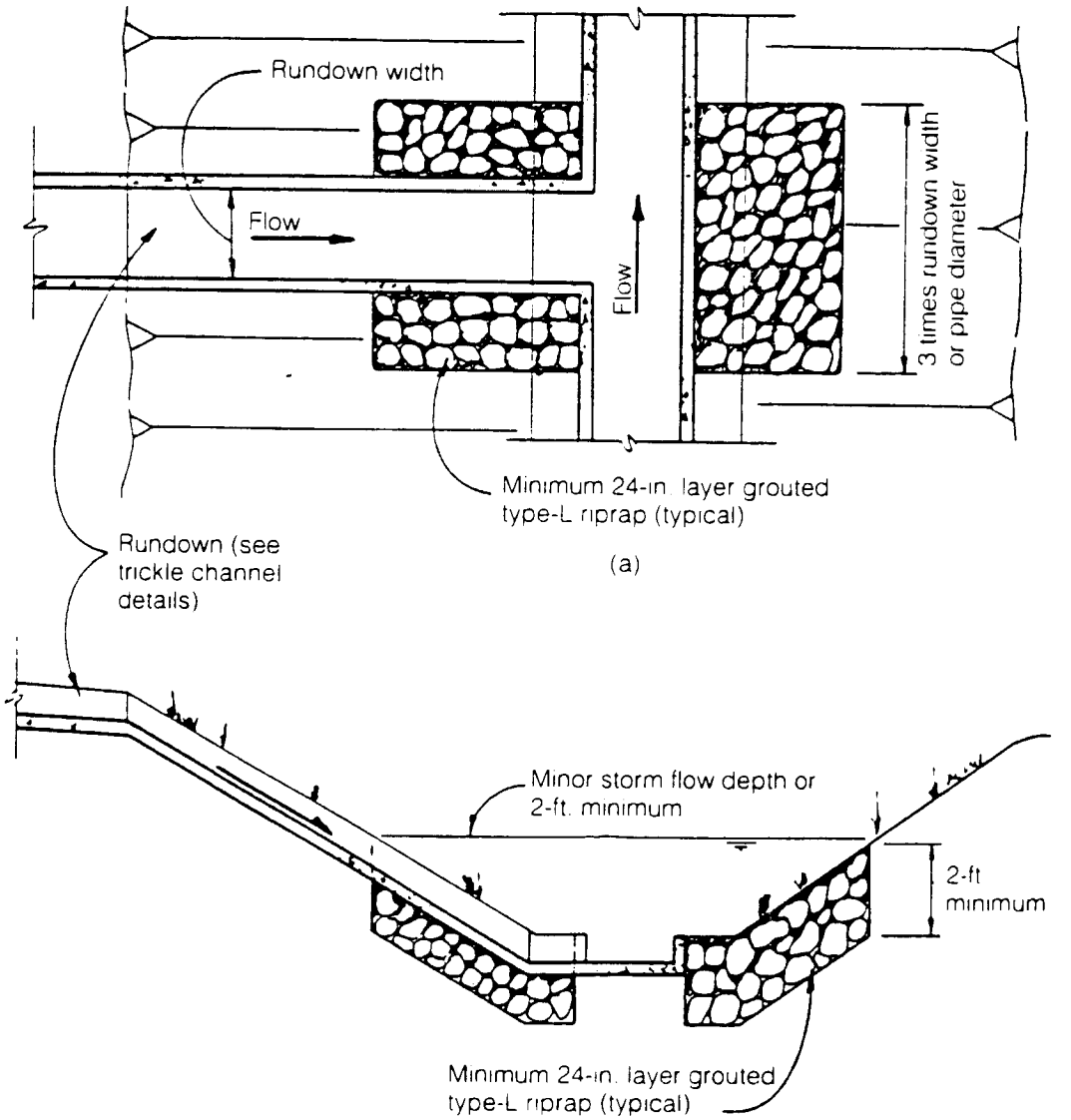

Figure 8.20—Channel rundown details: (a) plan and (b) profile.

X. CULVERTS AND BRIDGES

A. Culverts

Much of the following text and figures on culverts has been adapted from Federal Highway Administration publications (U.S. Department of Commerce 1965a and b; U.S. Department of Transportation 1972 and 1985). The reader should consult these references for more detail. The type of culvert used depends on such factors as roadway profiles, channel characteristics, flood damage evaluations, construction and maintenance costs, estimates of service life and public safety considerations.

Culverts are constructed from a variety of materials and are available in many different shapes and configurations (see Figure 8.21). The shape selection is based on construction costs, limitations on upstream water surface elevation, roadway embankment height and hydraulic performance. The selection of a culvert material depends upon its required structural strength, hydraulic roughness, durability, and corrosion and abrasion resistance. The three most common culvert materials are concrete (reinforced and non-reinforced), corrugated aluminum and corrugated steel. Culverts may also be lined with other materials to inhibit corrosion and abrasion or to reduce hydraulic resistance.

Many different inlet configurations are used on culvert barrels. They can either be prefabricated or constructed in place, and common inlet configurations include projecting culvert barrels, cast-in-place concrete headwalls, precast or prefabricated end sections, and culvert ends mitered to conform to the fill slope (see Figure 8.22). Structural stability, aesthetics, erosion control, and other factors influence the selection of various inlet configurations.

The hydraulic capacity of a culvert may be improved by appropriate inlet selection. Since the natural channel is usually wider than the culvert barrel, the inlet is a flow contraction and may be the primary flow control. The provision of a more gradual flow transition will reduce energy loss and create a more hydraulically efficient inlet condition. A flowchart of the desirable culvert design procedure is provided in Figure 8.23.

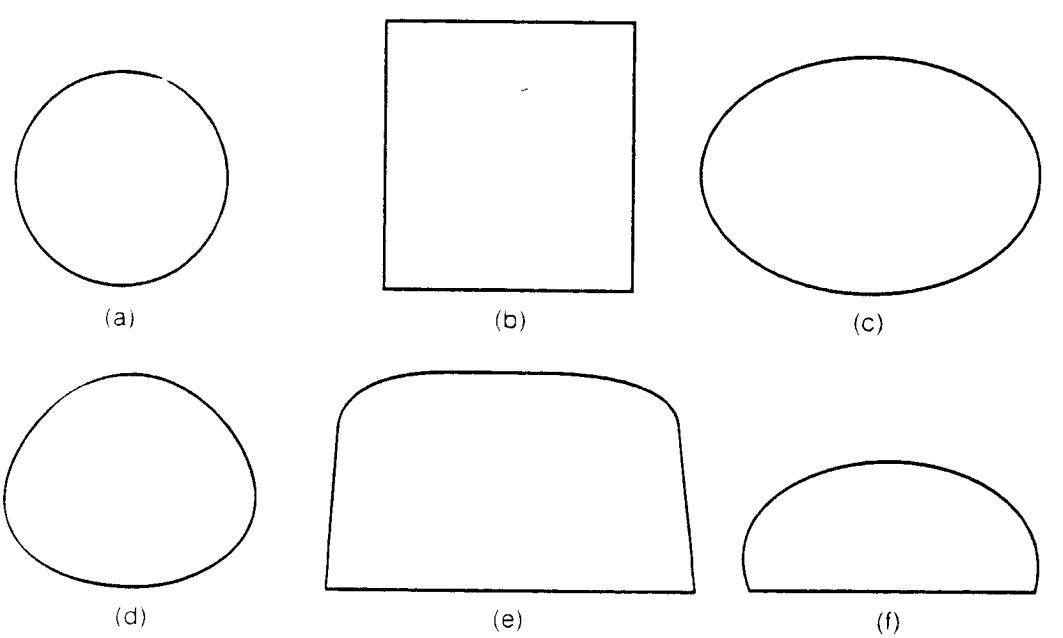

Figure 8.21—Commonly used culvert shapes: (a) circular, (b) box (rectangular), (c) elliptical, (d) pipe arch, (e) metal box, and (f) arch (U.S. Dept. of Transportation, 1985).

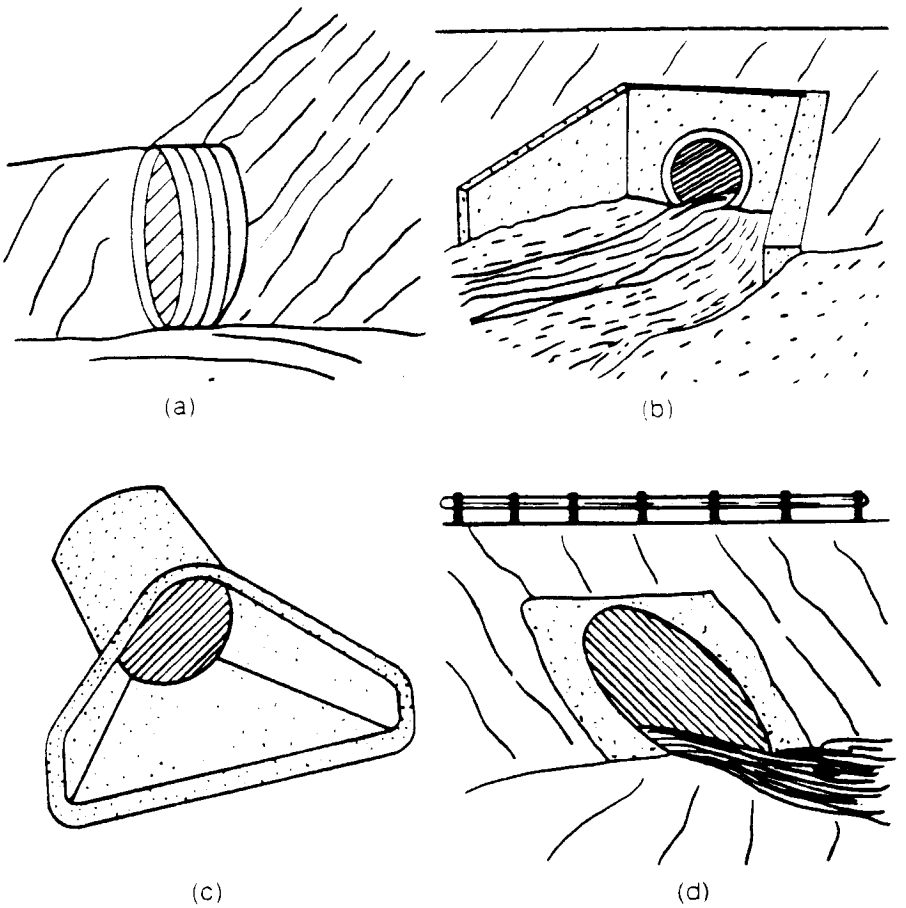

Figure 8.22—Four standard inlet types (schematic): (a) projecting barrel, (b) cast-in-place concrete headwall and wingwalls, (c) precast end section, and (d) end mitered to the slope (U.S. Dept. of Transportation, 1985).

1. *Culvert Hydraulics*

Culverts can flow under inlet control or outlet control (see also Chapter 6). Under inlet control, the cross-sectional area of the barrel, the inlet configuration or geometry, and the amount of headwater are the factors affecting capacity. Outlet control involves the additional consideration of the tailwater in the outlet channel and the slope, roughness, and length of barrel. Under inlet control conditions, the slope of the culvert is steep enough so that the culvert does not flow full and tailwater does not affect the flow.

Nomographs have been used extensively in culvert design. There are a number of easy to use computer programs available now which replace these nomographs.

Inlet Control Condition Inlet control for culverts may occur in two ways (see Figure 8.24).

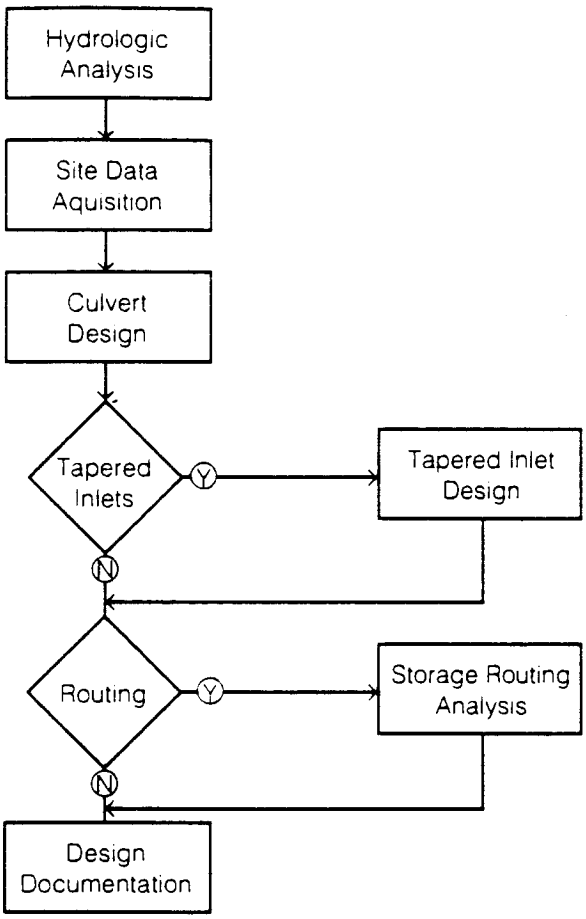

Figure 8.23—Culvert design procedure flow chart (U.S. Dept. of Transportation, 1985).

(a) Unsubmerged—the headwater is not sufficient to submerge the top of the culvert and the culvert invert slope is steep (can sustain supercritical flow). The culvert inlet effectively acts like a weir (Condition A, Figure 8.24.
(b) Submerged—the headwater submerges the top of the culvert but the pipe does not flow full. The culvert inlet acts like an orifice (Condition b, Figure 8.24).

In the submerged inlet condition, the equation governing the culvert capacity is the orifice flow equation:

$$Q = C_d A \sqrt{2gh} \tag{8-4}$$

where

Q = flow.
C_d = orifice coefficient.
A = area.
g = gravitational constant.
h = head on culvert measured from pipe/barrel centerline.

The orifice coefficient, C_d, varies with head on the culvert as well as the culvert type and entrance geometry. Inlet control rating curves for several culvert materials, shapes, and inlet configurations are given in Figures 8.25 and 8.26. These nomographs were developed empirically by the pipe manufacturers, Bureau of Public Roads, and the Federal Highway Administration. The nomographs (or computer programs that replace them) are recommended for use, rather than Equation 8-4, due to the uncertainty in estimating the orifice coefficient, and because they are applicable to a wide range of depths. Equation 8-4 is applicable only when the ratio of water depth to culvert height (diameter) is $\geqslant 2$.

Outlet Control Condition Outlet control will govern if the headwater is deep enough, the culvert slope sufficiently flat, and the culvert sufficiently long. There are three types of outlet control flow conditions:

(a) The headwater submerges the culvert top, and the culvert outlet is submerged under the tailwater. The culvert will flow full (Condition a, Figure 8.24).
(b) The headwater submerges the top of the culvert and the culvert is unsubmerged by the tailwater (Condition b or c, Figure 8.24).
(c) The headwater is insufficient to submerge the top of the culvert. The culvert slope is subcritical and the tailwater depth is lower than the pipe critical depth (Condition d, Figure 8.24).

The factors affecting the capacity of a culvert in outlet control include the inlet geometry and associated losses, the culvert material with friction losses, and the tailwater condition.

The capacity of the culvert under outlet control is calculated using the principle of the conservation of energy (Bernoulli's Equation). An energy balance is determined between the headwater at the culvert inlet and at the culvert outlet, which includes inlet losses, friction losses, and the velocity head (see Figure 8.27). The equation is then expressed as:

$$H = h_e + h_f + h_v \tag{8-5}$$

where

H = total energy head (feet).
h_e = entrance head losses (feet).
h_f = friction losses (feet).
h_v = velocity head (feet) = $V^2/2g$.

For entrance losses the governing equation is:

$$h_e = K_e \left(\frac{V^2}{2g}\right) \tag{8-6}$$

where K_e = is the entrance loss coefficient.

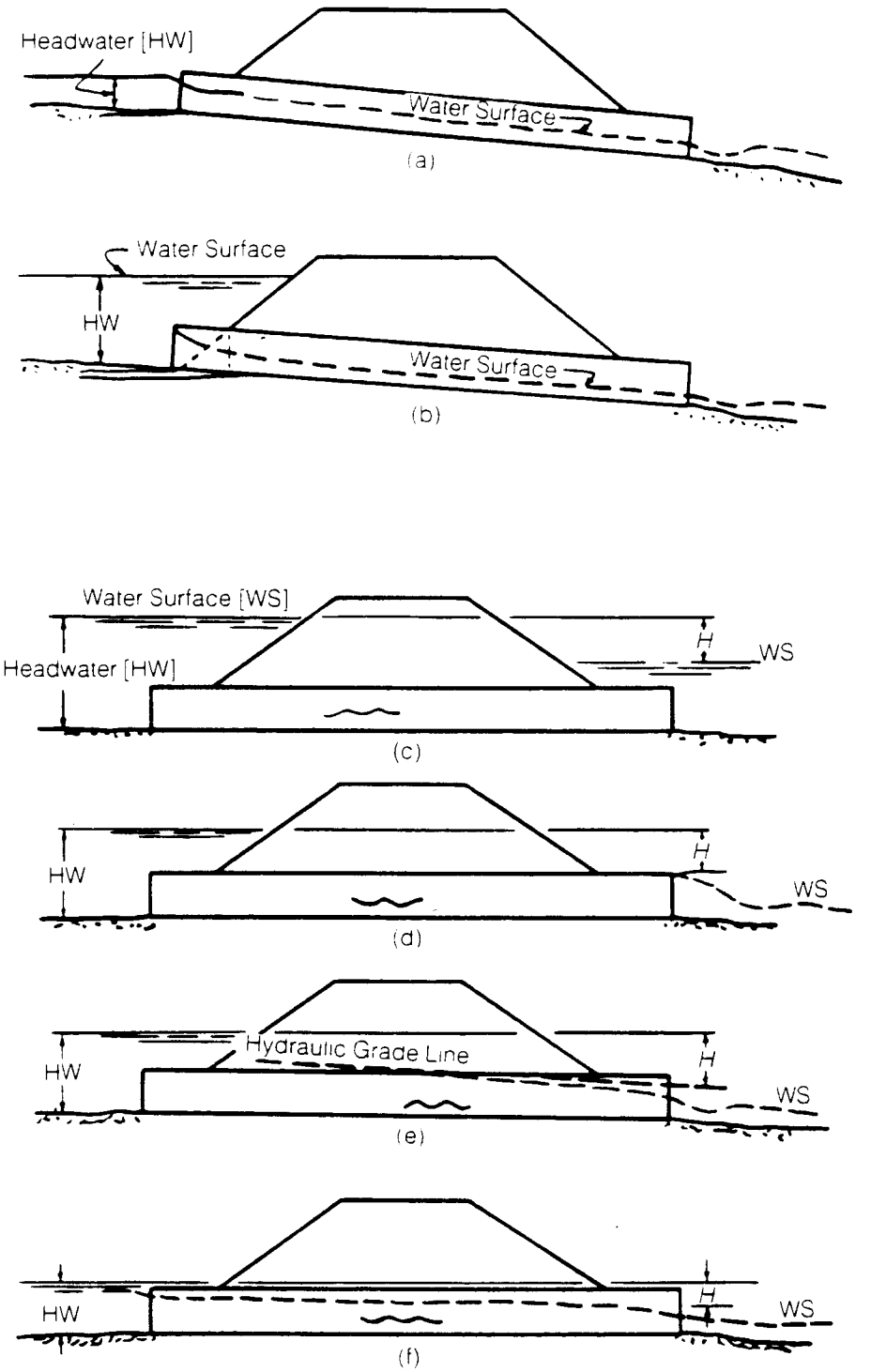

Figure 8.24—Inlet and outlet conditions for culverts: inlet control: (a) projecting inlet end—unsubmerged, (b) projecting or mitered inlet—submerged, and outlet control: (c), (d), (e), and (f) (see text for explanation) (U.S. Dept. of Commerce, 1965 a & b).

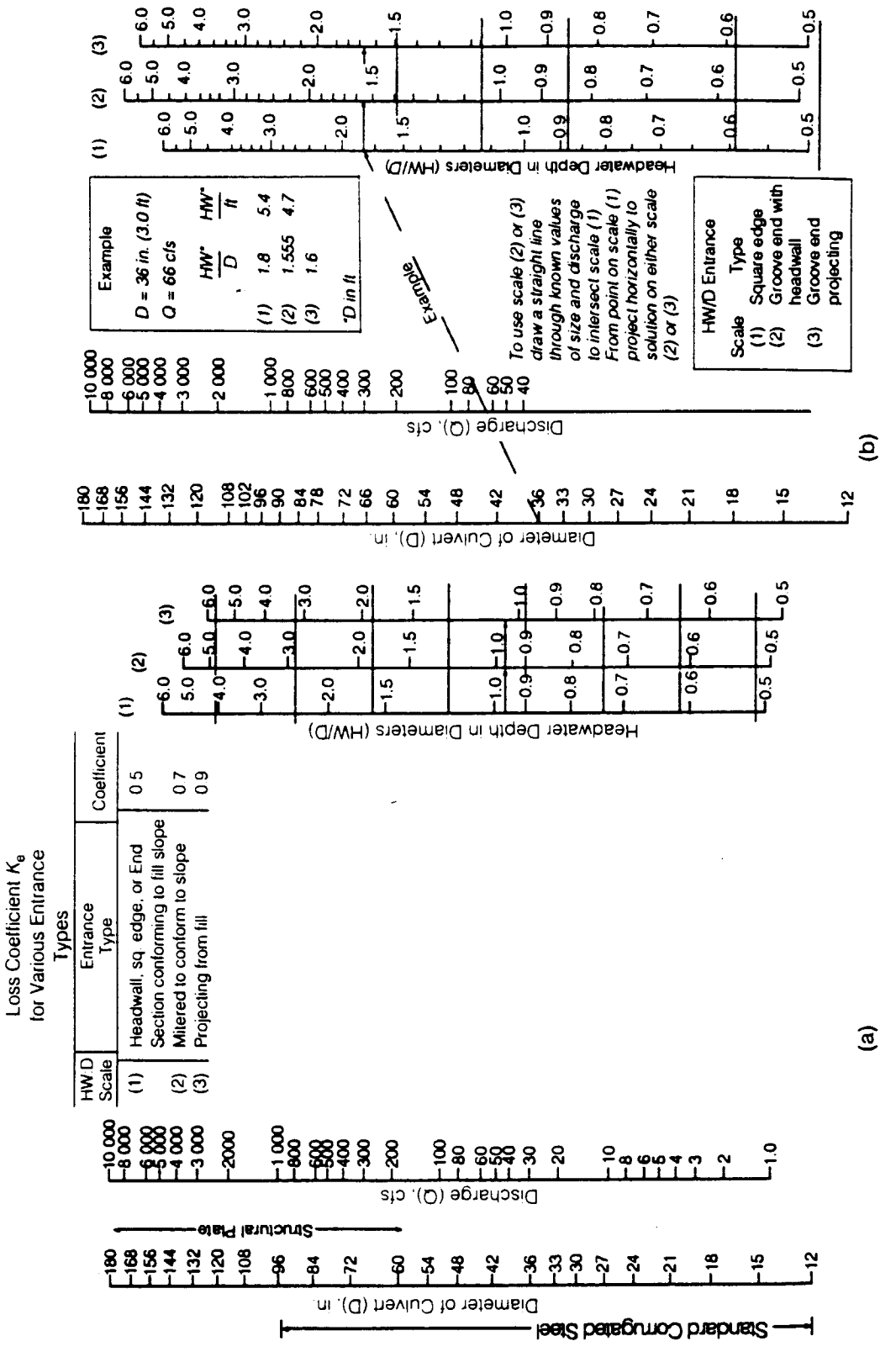

Figure 8.25—Inlet control nomograph circular pipe.

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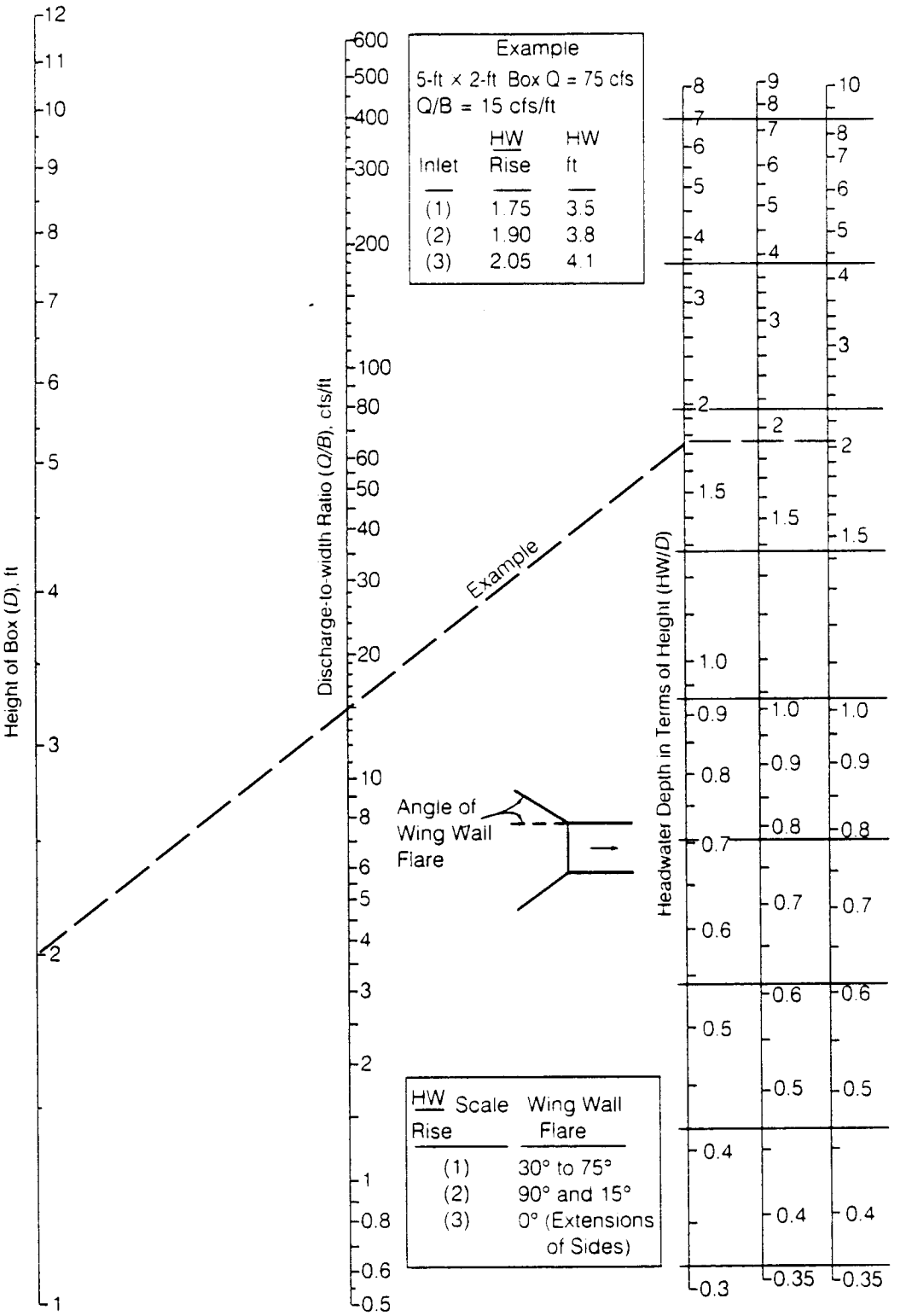


Figure 8.26—Inlet control box culverts nomograph (AISI, 1983 and U.S. Bureau of Public Roads, 1963 a & b).

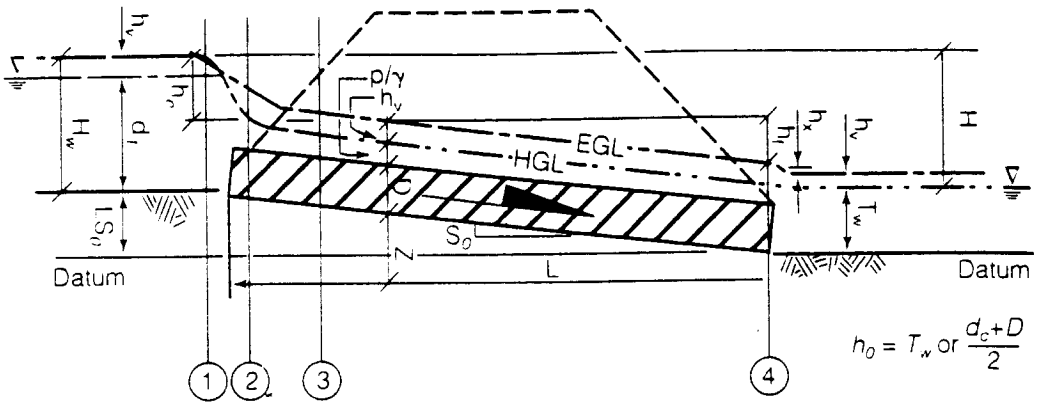


Figure 8.27—Hydraulics of a culvert under outlet condition: L = culvert length, S_o = culvert slope, H_w = headwater depth, h_v = velocity head, h_e = head loss at entrance, Z = distance from datum line, D = culvert diameter or rise, p/γ = pressure head, HGL = hydraulic grade line, EGL = energy grade line, T_w = tailwater depth, h_x = head loss at exit, and h_f = friction loss in culvert (U.S. Dept. of Transportation, 1985).

Typical inlet loss coefficients recommended for use are given in Table 8.8.

Friction loss is the energy required to overcome the roughness of the culvert and is expressed as follows:

$$h_f = \frac{29n^2L}{R^{1.33}}\left(\frac{V^2}{2g}\right) \tag{8-7a}$$

and, in SI units,

$$h_f = \frac{19.5n^2L}{R^{1.33}}\left(\frac{V^2}{2g}\right) \tag{8-7b}$$

where

n = Manning's coefficient.
L = length of culvert.
R = hydraulic radius.

Combining Equations 8-5, 8-6, and 8-7 and simplifying gives:

$$H = \frac{K_e + 1 + 29n^2L}{R^{1.33}}\left(\frac{V^2}{2g}\right) \tag{8-8a}$$

and, in SI units,

$$H = \frac{K_e + 1 + 19.5n^2L}{R^{1.33}} \left(\frac{V^2}{2g}\right) \tag{8-8b}$$

Equation 8-8 can be used to calculate the culvert capacity directly when the culvert is flowing under outlet conditions A or B as shown on Figure 8.24. For conditions C or D, the HGL at the outlet is approximated by averaging the critical depth and the culvert diameter, which is used if the value is greater than the tailwater depth (Tw) to compute headwater depth (Hw).

A series of outlet control nomographs for various culvert materials and shapes have also been developed. The nomographs are presented in Figures 8.28, 8.29, and 8.30 (computer programs are also available that greatly simplify the calculations). When rating a culvert, either the outlet control nomographs or Equation 8-8 can be used to calculate the headwater requirements.

When using the outlet nomographs for corrugated steel pipe the data must be adjusted to account for the variation in the n-value between the nomographs and the culvert being evaluated. The adjustment is made by calculating an equivalent length according to the following equation:

$$L' = L\left(\frac{n'}{n}\right)^2 \tag{8-9}$$

where

L' = equivalent length.
L = actual length.
n = Manning's n.
n' = actual n-value of culvert.

2. *Design Procedure and Example*

Computer programs (proprietary programs, available from several vendors) suitable for use on a microcomputer are readily available, and are easy to use. If a computer is not available, the following detailed procedure for selection of culvert size is adopted directly from Hydraulic Engineering Circular No. 5: Hydraulic Charts for the Selection of Highway Culverts (U.S. Department of Commerce 1965a).

Design Example

Step 1: List design data. (see suggested tabulation form, Table 8.9).

(a) Design discharge Q, in cfs, with average return period (i.e., Q_{25} or Q_{50}, etc.).
(b) Approximate length L of culvert, in feet.

TABLE 8.8. Hydraulic Data for Culvert: Culvert Entrance Losses (UDFCD 1984).

Type of Entrance	Entrance Coefficient, K_e
Pipe	
Headwall	
Grooved edge	0.20
Rounded edge (0.15D radius)	0.15
Rounded edge (0.25D radius)	0.10
Square edge (cut concrete and CMP)	0.40
Headwall & 45° Wingwall	
Grooved edge	0.20
Square edge	0.35
Headwall with Parallel Wingwalls Spaced 1.25D apart	
Grooved edge	0.30
Square edge	0.40
Beveled edge	0.25
Projecting Entrance	
Grooved edge (RCP)	0.25
Square edge (RCP)	0.50
Sharp edge, thin wall (CMP)	0.90
Sloping Entrance	
Mitered to conform to slope	0.70
Flared-end Section	0.50
Box, Reinforced Concrete	
Headwall Parallel to Embankment (no wingwalls)	
Square edge on 3 edges	0.50
Rounded on 3 edges to radius of 1/12 barrel dimension	0.20
Wingwalls at 30° to 75° to barrel	
Square edged at crown	0.40
Crown edge rounded to radius of 1/12 barrel dimension	0.20
Wingwalls at 10° to 30° to barrel	
Square edged at crown	0.50
Wingwalls parallel (extension of sides)	
Squared edged at crown	0.70

NOTE: The entrance loss coefficients are used to evaluate the culvert or sewer capacity operating under outlet control.

(c) Slope of culvert. (If grade is given in percent, convert to slope in feet per foot).
(d) Allowable headwater depth, in feet, which is the vertical distance from the culvert invert (flow line) at the entrance to the water surface elevation permissible in the headwater pool or approach channel upstream from the culvert.

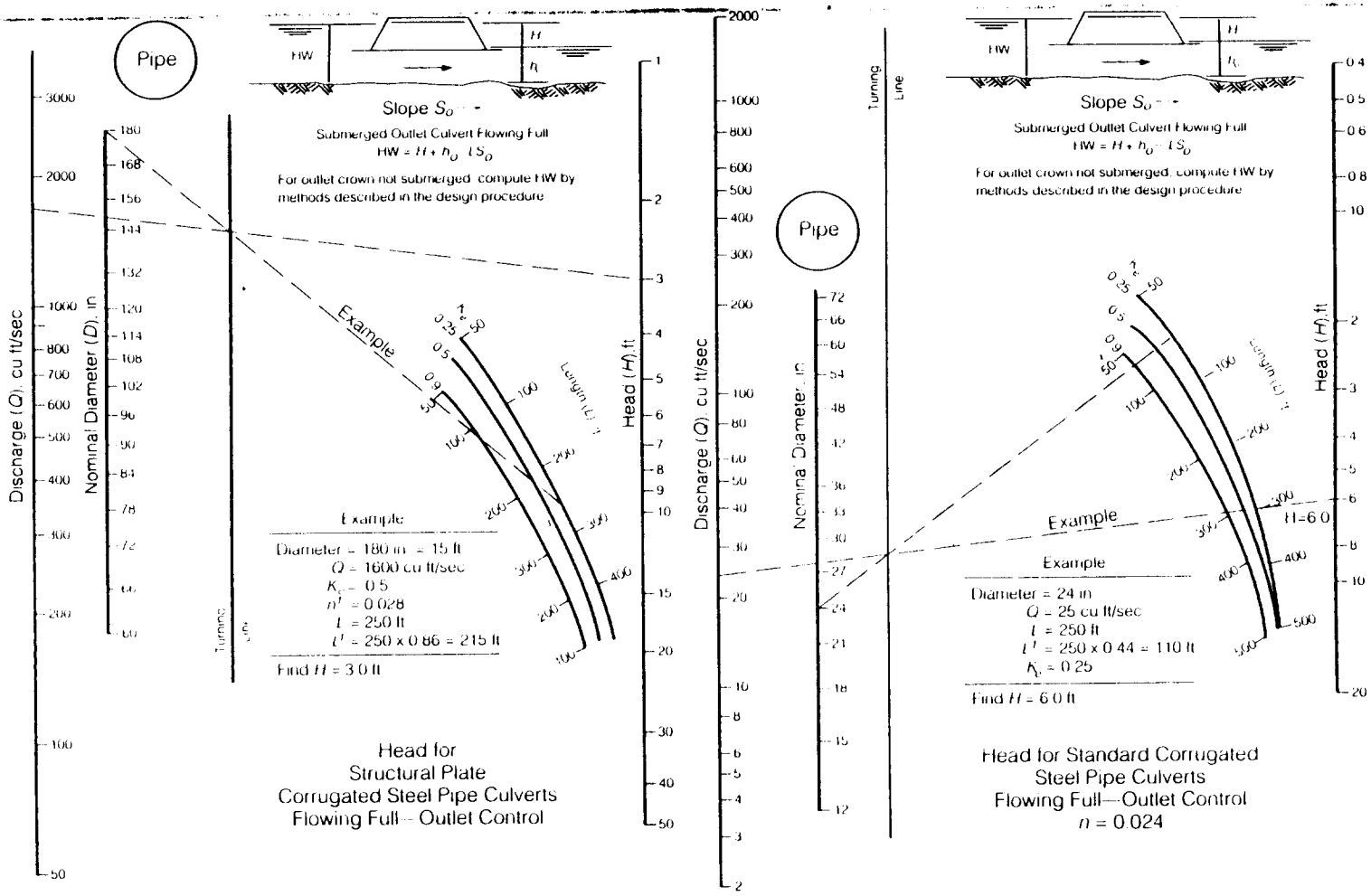


Figure 8.28—Outlet control nomograph circular CSP. Note: See AISI, 1983, for length adjustment for pipe diameters larger than 10.0 ft. (in. × 25.4 = mm, cu ft/sec × 0.028 32 = m^3/s, and ft × 0.304 8 = m) (AISI, 1983).

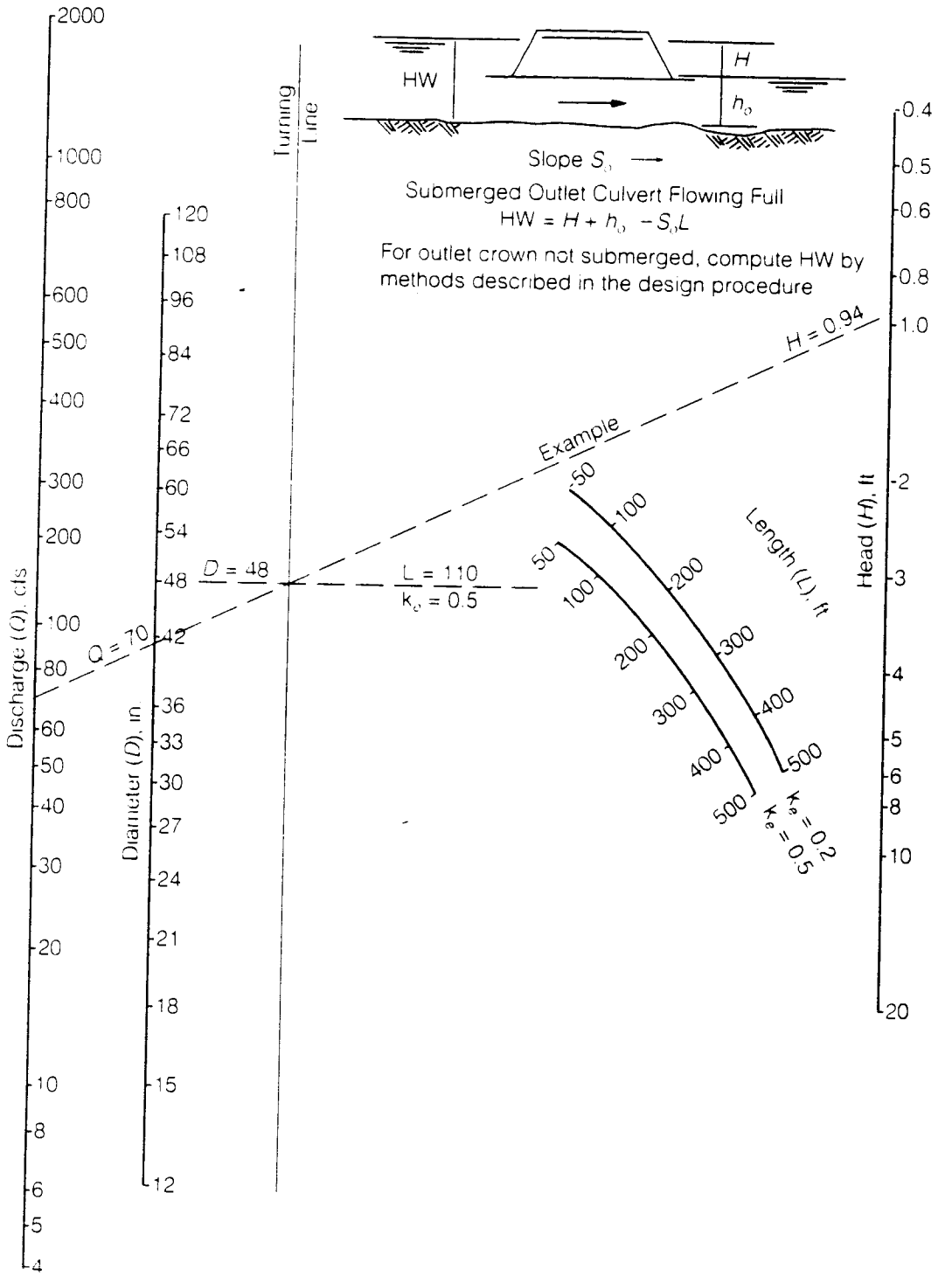


Figure 8.29—Outlet control nomograph circular RCP (cfs × 0.028 32 = m^3/s, ft × 0.304 8 = m, and in. × 25.4 = mm) (U.S. Dept. of Commerce, 1965 a and b).

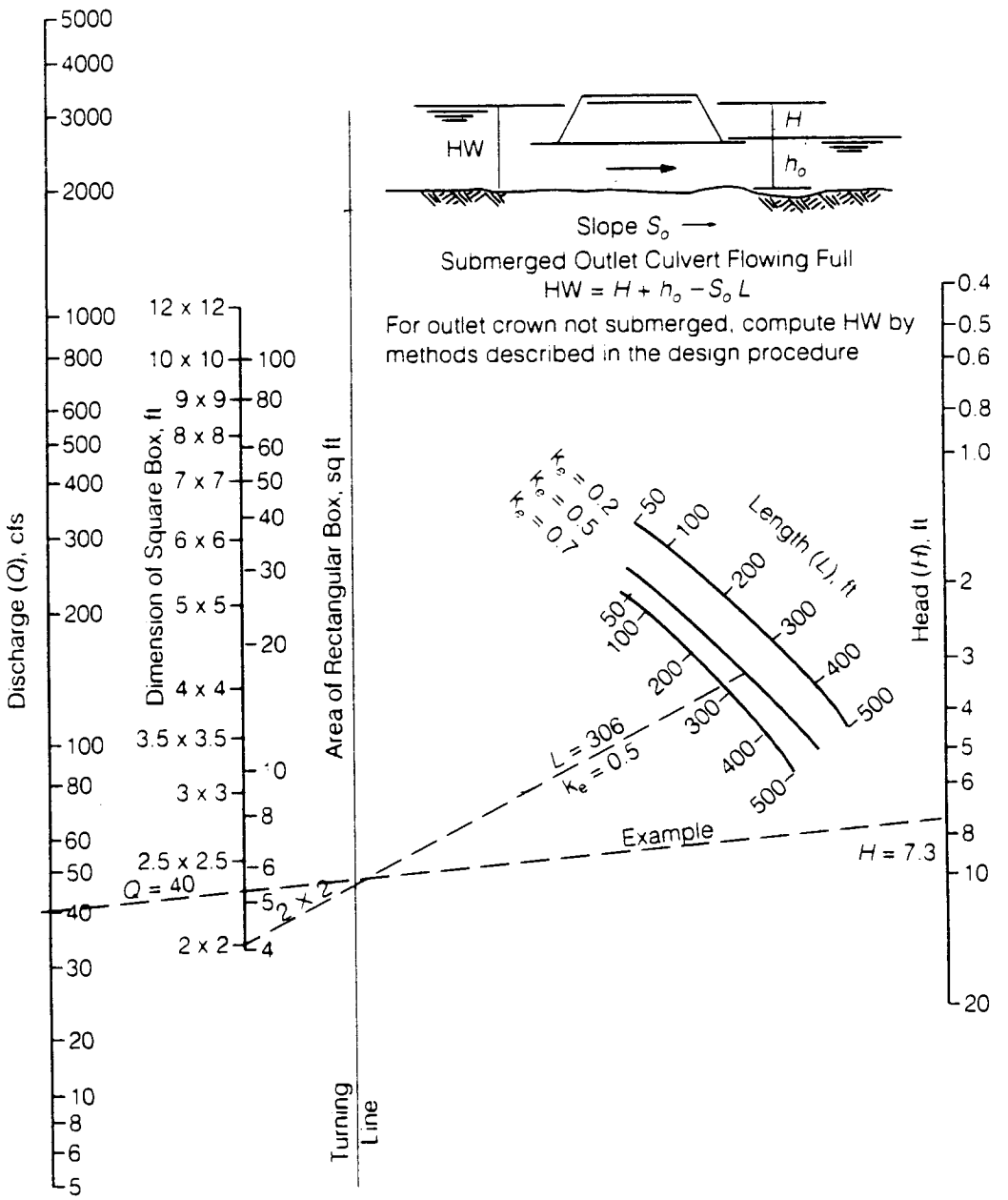


Figure 8.30—Outlet control nomograph box culverts (cfs × 0.028 32 = m^3/s, ft × 0.304 8 = m, and sq ft × 0.092 9 = m^2 (U.S. Dept. of Commerce, 1965 a and b).

TABLE 8.9. Culvert Design Form (UDFCD 1984)

PROJECT: _____

DESIGNER: _____

DATE: _____

HYDROLOGIC AND CHANNEL INFORMATION

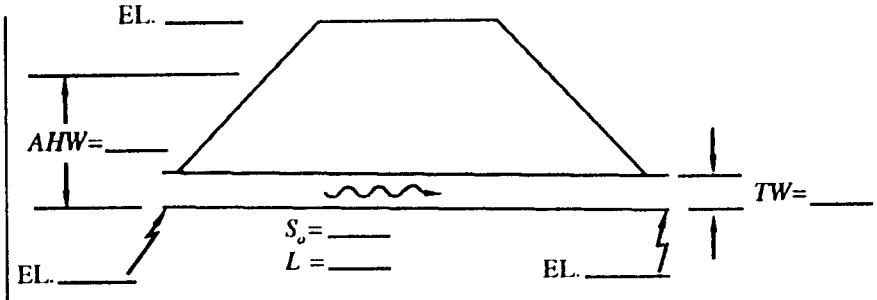
Q_1 = _____ TW_1 = _____

Q_2 = _____ TW_2 = _____

(Q_1 = DESIGN DISCHARGE, SAY Q_{25} OR Q_{100}
Q_2 = CHECK DISCHARGE, SAY Q_{50})

SKETCH

STATION: _____



MEAN STREAM VELOCITY = _____

MAX. STREAM VELOCITY = _____

Culvert Description (Entrance type)	Q	Size	Headwater Computation										Controlling HW	Outlet Velocity	Cost	Comments
			Inlet Cont.		Outlet Control ($HW = H + h_o - LS_o$)											
			$\frac{HW}{D}$	HW	K_e	H	d_c	$\frac{d_c + D}{2}$	TW	h_o	LS_o	HW				

Summary & Recommendations:

(e) Mean and maximum flood velocities in natural stream.
(f) Type of culvert for first trial selection, including barrel material, barrel cross-sectional shape and entrance type.

Step 2: Determine the first trial size culvert

Since the procedure given is one of trial and error, the initial trial size can be determined in several ways:

(a) By arbitrary selection.
(b) By using an approximating equation such as $Q/10 = A$ from which the trial culvert dimensions are determined.
(c) By using inlet control nomographs (Figures 8.26 or 8.27) for the culvert type selected. If this method is used an HW/D must be assumed, say HW/D = 1.5, and using the given Q a trial size is determined.

If any trial size is too large because of limited embankment height or availability of pipe, multiple culverts may be used by dividing the discharge equally among the number of barrels used. Raising the embankment height or the use of pipe arch and box culverts with width greater than height should be considered. Final selection should be based on an economic analysis.

Step 3: Find headwater depth for trial size culvert

(a) Assuming INLET CONTROL.
   1. Using the trial size from step 2, find the headwater depth, HW, by use of the appropriate inlet control nomograph. Tailwater (TW) conditions are to be neglected in this determination. HW in this case is found by multiplying HW/D obtained from the nomographs by the height of culvert D.
   2. If HW is greater or less than allowable, try another trial size until HW is acceptable for inlet control before computing HW for outlet control.
(b) Assuming OUTLET CONTROL.
   1. Approximate the depth of tailwater TW, in feet, above the invert at the outlet for the design flood condition in the outlet channel.
   2. For tailwater TW elevation equal to or greater than the top of the culvert at the outlet, set h_o equal to TW and find HW by the equation given in Table 8.10.
   3. For tailwater TW elevation less than the top of the culvert at the outlet, find headwater HW by the equation given in Table 8.10, except that:

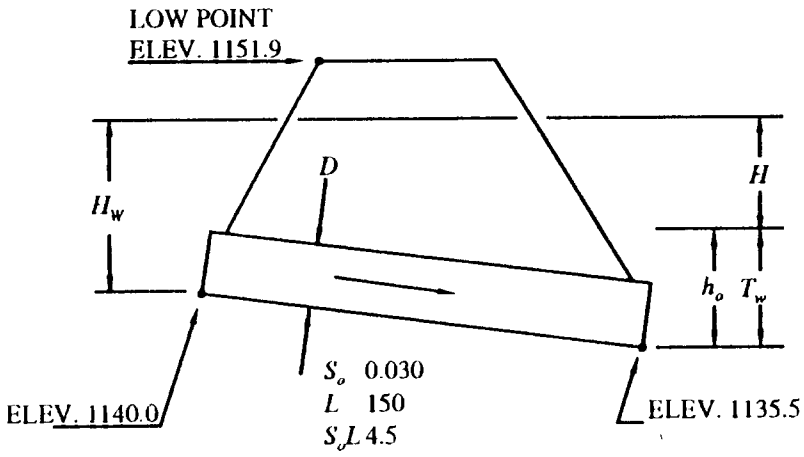
$$h_o = \frac{d_c + D}{2}$$

or TW, whichever is the greater, where:

d_c = critical depth in feet
D = height of culvert opening, in feet

TABLE 8.10. Culvert Rating Form (City of Norman 1989)

PROJECT: _____ LOCATION: _____ STATION: _____



CULVERT DATA

TYPE:	48 in. CMP	n:	0.024
INLET:	Flared end section	Q_{FULL}:	135
K_e:	0–5	V_{FULL}:	10.7

OUTLET CONTROL EQUATIONS

(1) $H_W = H + h_o - LS_o$

(2) For $T_W < D$, $h_o = \frac{d_c + D}{2}$ or T_w (whichever is greater)
$T_w > D$, $h_o = T_w$

(3) For Box Culvert: $d_c = 0.315(Q/B)^{2/3} \leq D$

	Inlet Control		Outlet Control								
					$T_W < D$		$T_W > D$				
Q	$\frac{H_W}{D}$	H_W	H	T_W	d_c	$\frac{d_c + D}{2} = h_o$	h_o	H_W	Cont. H_W	Control	Elev.
1	2	3	4	5	6	7	8	9	10	11	12
70	1.0	4	1.9	1.5	2.5	3.3		0.7	4	Inlet	1144.0
115	1.5	6	5.5	2.0	3.0	3.5		4.5	6	Inlet	1146.0
145	2.0	8	8.9	2.5	3.4	3.7		8.1	8.1	Outlet	1148.8
170	2.5	10	12.5	3.0	3.7	3.9		11.9	11.9	Outlet	1151.9
195	3.0	12	16.0	3.5	4.0	4.0		15.5	15.5	Outlet	1155.5

Outlet Velocity, $V = Q/A = 170$ cfs/12.6 ft^2 = 13.5 fps

(c) Compare the headwaters found in Step 3a and Step 3b (Inlet Control and Outlet Control). The higher headwater governs and indicates the flow control existing under the given conditions for the trial size selected.
(d) If outlet control governs and the HW is higher than is acceptable, select a larger trial size and find HW as instructed under Step 3b. (Inlet control need not be checked, since the smaller size was satisfactory for this control as determined under Step 3a.)

Step 4: Try a culvert of another type or shape and determine size and HW by the above procedure.

Step 5: Compute outlet velocities for size and types to be considered in selection and determine need for channel protection.

(a) If outlet control governs in Step 3c above, outlet velocity equals Q/A_o, where A_o is the cross-sectional area of flow in the culvert barrel of the outlet. If d_c or TW is less than the height of the culvert barrel, use A_o corresponding to d_c or TW depth, whichever gives the greater area of flow. A_o should not exceed the total cross-sectional area A of the culvert barrel.
(b) If inlet control governs in Step 3c, outlet velocity can be assumed to equal mean velocity in open channel flow in the barrel as computed by Manning's equation for the rate of flow, barrel size, roughness and slope of culvert selected.
Note: Charts and tables are helpful in computing outlet velocities (see references noted in the first paragraph of this section).

Step 6: Record final selection of culvert with size, type, required headwater, outlet velocity, and economic justification.

Step 7: (Not contained in reference). Assess flood hazard posed by recommended culvert to upstream and downstream properties, particularly if a change in the status quo, such as increased flood levels, is going to occur. Have client and review authorities make an informed decision as to appropriate culvert size.

Design Example—Rating an Existing Culvert A sample calculation for rating an existing culvert is presented in Table 8.10. The required data are as follows:

(a) Culvert size, length, and type (48" CMP, L = 150, n = .024).
(b) Inlet, outlet elevation, and slope (1140.0, 1135.5, S_o = 0.030).
(c) Inlet treatment (flared end section).
(d) Low point elevation of embankment (elevation = 1151.9).
(e) Tailwater rating curve (see Table 8.10, Column 5).

From the above data, the entrance loss coefficient, K_e, (Table 8.8) and the n-value are determined. The full flow Q and the velocity are calculated from these values for comparison. The rating then proceeds in the following sequence:

Step 1: Headwater values are selected and entered in Column 3. The headwater to pipe diameter ratio (HW/D) is calculated and entered in Column 2. If the culvert is not circular, the height of the culvert is used.

Step 2: For the HW/D ratios, the inlet rating is read from the various figures based upon culvert type (see Figure 8.26 for the example) and entered into Column 1. This completes the inlet condition rating portion.

Step 3: For outlet condition, the Q values in Column 1 are used to determine the head values (H) in Column 4 from the appropriate outlet rating curves (see Tables 8.9 and 8.10 for examples).

Step 4: The tailwater depths (TW) are entered into Column 5 for the corresponding Q values in Column 1 according to the tailwater rating curve. If a tailwater rating curve is not available, compute the normal depth (subcritical or critical only) of a trapezoidal channel approximating the existing drainageway. If the tailwater depth (TW) is less than the diameter of the culvert (d), Columns 6 and 7 are to be calculated (go to Step 5). If TW > d, the tailwater values in Column 5 are entered into Column 8 for the h_o values and proceed to Step 6.

Step 5: The critical depth (d_c) for the corresponding Q values in Column 1 are read from appropriate figures or computed and entered into Column 6. The average of the critical depth and the culvert diameter is calculated and entered into Column 7 as the h_o values.

Step 6: The headwater values (HW) are calculated according to the equation:

$$H_w = H + h_o - LS_o \tag{8-10}$$

where H is from Column 4 and h_o is the value from Column 8 (for Tw > D) or the larger value between Column 5 and Column 7 (for Tw < D). The values are entered into Column 9.

Step 7: The final step is to compare the headwater requirements (Column 9 and 3) and to record the higher of the two values in Column 10. The type of control is recorded in Column 11, depending upon which case gives the higher headwater requirements. The headwater elevation is calculated by adding the controlling H_w (Column 10) to the upstream invert elevation. A culvert rating curve can then be plotted from the values in Column 12 and 1.

Step 8: Compute outlet velocity for designing downstream protection. For full flow conditions, $V = Q/A$; for partially full conditions, see Figure 6.13 for hydraulic properties of pipe. Design channel protection as described in Chapter 9.

To size a culvert crossing, the same form can be used, with some variation in the basic data. First, a design Q value is selected and the maximum allowable headwater is determined. An inlet type (i.e., headwall) is selected and the invert elevations and culvert slope are estimated based upon site constraints. A culvert type is then selected and first rated for inlet control then outlet control. If the controlling headwater exceeds the maximum allowable headwater, the input data are modified and the procedure repeated until the desired results are achieved.

3. *Design Considerations*

Culvert Sizing Factors to be considered in the sizing of the culvert include, but are not limited to: (1) minimum design frequency, (2) minimum culvert size, (3) allowable cross street flow in the street for which the culvert is being considered, (4) public safety, (5) maintenance, (6) allowable frequency of inundation of upstream and downstream properties, and (7) potential for debris accumulation.

The minimum design frequency for culverts is often specified by such entities as city or county governments, state highway departments, or the Federal Highway Administration. Frequency often is stated in terms of a maximum headwater depth for a given return frequency flood. It is essential for the engineer to recognize that the design frequency must be selected for each culvert on a case-by-case basis; a 10-year culvert may be acceptable under certain conditions, whereas a culvert with much greater than 100-year capacity may be required in others. Recognition of the nature of development both upstream and downstream is of foremost importance. For instance, a nuclear power facility or hospital immediately upstream from a location requiring culverts would argue for a conservative design. By contrast, if the engineer has assurance that upstream land will stay undeveloped for decades and if periodic inundation of the subject land will not otherwise be harmful (or if the developer has obtained the rights for backwater storage), then a culvert with only nominal capacity could suffice. In any event, the engineer should obtain review authority and client approval of his tentative selection of design return frequency.

Localities often state the minimum culvert size (commonly in the 12–24 inch range). Debris potential of the tributary basin is an important consideration when determining the minimum acceptable size for a culvert. A viable alternative to increased culvert capacity may be to lengthen the roadway area subject to overflow.

When the flow in a channel exceeds the capacity of the culvert and overtops the cross street, the flow across the street must not exceed stipulated local inundation criteria. If the cross street flow exceeds the limits for the minimum design frequency or the minimum culvert size, then the culvert must be increased in size until all criteria are met.

The debris potential of the tributary basin, along with whether or not the culvert will have a debris/safety rack, are critical considerations when determining minimum size of the pipe. Localities often stipulate

that, for example, the engineer should assume 25% debris blockage when computing the required size of the culvert. Under certain conditions, greater than 25% debris blockage may be justified. Guidance on debris/safety rack installation follows in this section.

Velocity Limitations When designing culverts, both the minimum and maximum velocities should be considered. A minimum velocity of flow is required to assure self cleaning; this minimum velocity is normally taken to be between 2–3 feet per second at the outlet.

The maximum velocity in a culvert is controlled by two factors, the channel protection provided at the outlet and the maximum allowable headwater. If the outlet velocities are less than a stipulated value (based on site specific channel erosion factors) then only minimal protection is required due to the eddy currents generated by flow transitions. As outlet velocity is increased however, additional protection is required such as more extensive riprap or an energy dissipator structure. Discussion of outlet protection is provided in Chapter 9 of the Manual.

Headwater Depth Maximum headwater depths are normally stipulated by local governments, generally in terms of a particular factor times the culvert diameter or culvert rise dimension for shapes other than round. Considerations governing headwater selection are similar to those described above for design return frequency. Essentially, the engineer must carefully consider the consequences to upstream properties for a wide variety of floods for different headwater depths. Of particular importance is the historic precedent with respect to maximum headwater depths associated with the tributary area in question. Engineers are cautioned against down-sizing the conveyance capacity of a culvert for a given headwater depth in an attempt to save money. Such a change would increase flood risks to established upstream properties that have developed "reliance" (in a legal sense) on the existing conveyance capacity. Decreasing culvert capacity under these conditions leaves the engineer and his client potentially liable for damages in the event that a flood occurs that damages upstream properties due to the use of a replacement culvert, but would not have damaged the upstream properties had the original facility been in place. Finally, the design should consider the implications of the fact that a Federal District Court (Rooney et al. 1986) has defined an embankment with a culvert as a "dam" during times of flooding, with attendant hazard considerations.

Inlet and Outlet Protection Generally, the engineer should assume that inlet and outlet protection is required. Culvert inlets normally include a headwall with wingwalls or a flared end section. The outlet also typically includes a headwall with wingwalls or an end section in addition to riprap protection.

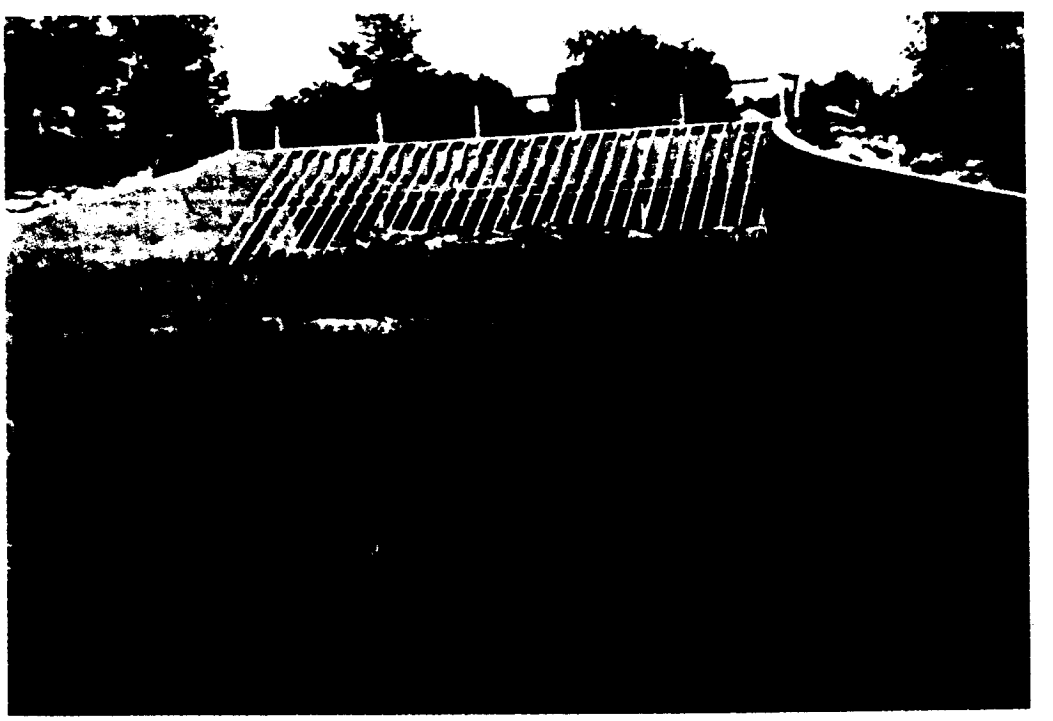
Structural Design All culverts should be designed to withstand minimum loading requirements in accordance with design procedures sum-

marized by the American Concrete Pipe Association in its publication "Standard Specifications for Highway Bridges" (Hendrickson 1957) and with pipe manufacturers' recommendations. Minimum depth of cover of culverts typically ranges from 1–3 feet.

4. *Debris/Safety Racks*

The engineer is obligated to consider the advantages and disadvantages of debris racks on a case-by-case basis. Guidance on the proper design of trash racks for detention pond outlet structures is found in "Stormwater Detention Outlet Control Structures" (ASCE 1985). Highlights from this guidance include the following:

(a) Trash racks should be hinged at the top to permit lifting and cleaning, and should slope at 3:1 to 5:1 (horizontal to vertical) to permit debris to "ride up" as the water level rises.
(b) The trash rack should be located a considerable distance out from the entrance to the culvert or pipe to assure that entrance velocities will be low enough that a person will be able to lift himself up.
(c) Bar spacing should be such that a child will not be able to pass between the bars (typically maximum 6 inches clear spacing).
(d) The net open surface area of the trash racks should be at least four times the cross-sectional area of the pipe.



Typical debris rack at a culvert. Such racks must be designed with low entrance velocities, and in a way that the debris floats upward as the water level rises. Access to the channel bottom must be provided to remove debris between flood events.

Culverts normally run short distances beneath roads, railroads, etc. In cases where drainage ditches transition to drainage pipes that run for hundreds or even thousands of feet (common in the Midwest), a safety rack at the pipe entrance is recommended because of the consequences of debris blockage within the pipe and for public safety.

B. Bridges

The designer of urban stormwater management systems will often find it necessary to characterize the hydraulic aspects of bridge crossings of major drainageways. Procedures described within this chapter with respect to culvert evaluation will often suffice. However, if a more rigorous evaluation is merited, special bridge evaluation techniques may be necessary. For a detailed discussion of this subject, the reader is referred to "Hydraulics of Bridge Waterways" (U.S. Department of Transportation 1970). The following discussion and figures regarding bridges are based upon edited excerpts from this publication. Other valuable bridge design references include the AISI Handbook of Steel Drainage and Highway Construction Products (1983), Neil et al. (1973), and "Hydrology for Transportation Engineers" (U.S. Department of Transportation 1980).

The bridge designer, even in comparatively small urbanized basins, will frequently find it economically advantageous to extend approach embankments into the floodplain to reduce construction costs, recognizing that, in so doing, the embankments will constrict the flow of the stream during flood stages. The greater the free span opening, the less the amount of backwater during the design flood, and vice versa. This has been considered acceptable practice for decades, so long as it is done within reason (Jones and Jones 1987).

When designing a new bridge or replacing an existing one, the designer is urged to conduct an economic evaluation of various bridge alternatives that accounts not only for the comparative construction costs of the bridges, but also includes corresponding flood damages to upstream properties posed by the alternatives. Such an evaluation permits the developer, municipality, or regulatory officials to make an informed decision as to:

(a) How much of a capital expenditure should be made on the bridge.
(b) Expected average annual flood damages for properties affected by the bridge.
(c) Whether or not it would be cost effective to purchase properties subject to backwater flooding upstream of the bridge to save on bridge construction costs (Federal Flood Insurance Administration 1981).

1. *Types of Flow Through Bridges*

Figure 8.31 provides plan and profile drawings for a typical bridge crossing of a major drainageway. Note the flow is divided into three regions, as labelled Q_a, Q_b, and Q_c. Q_b is that portion of the flow which is confined to the main channel and which passes through the bridge

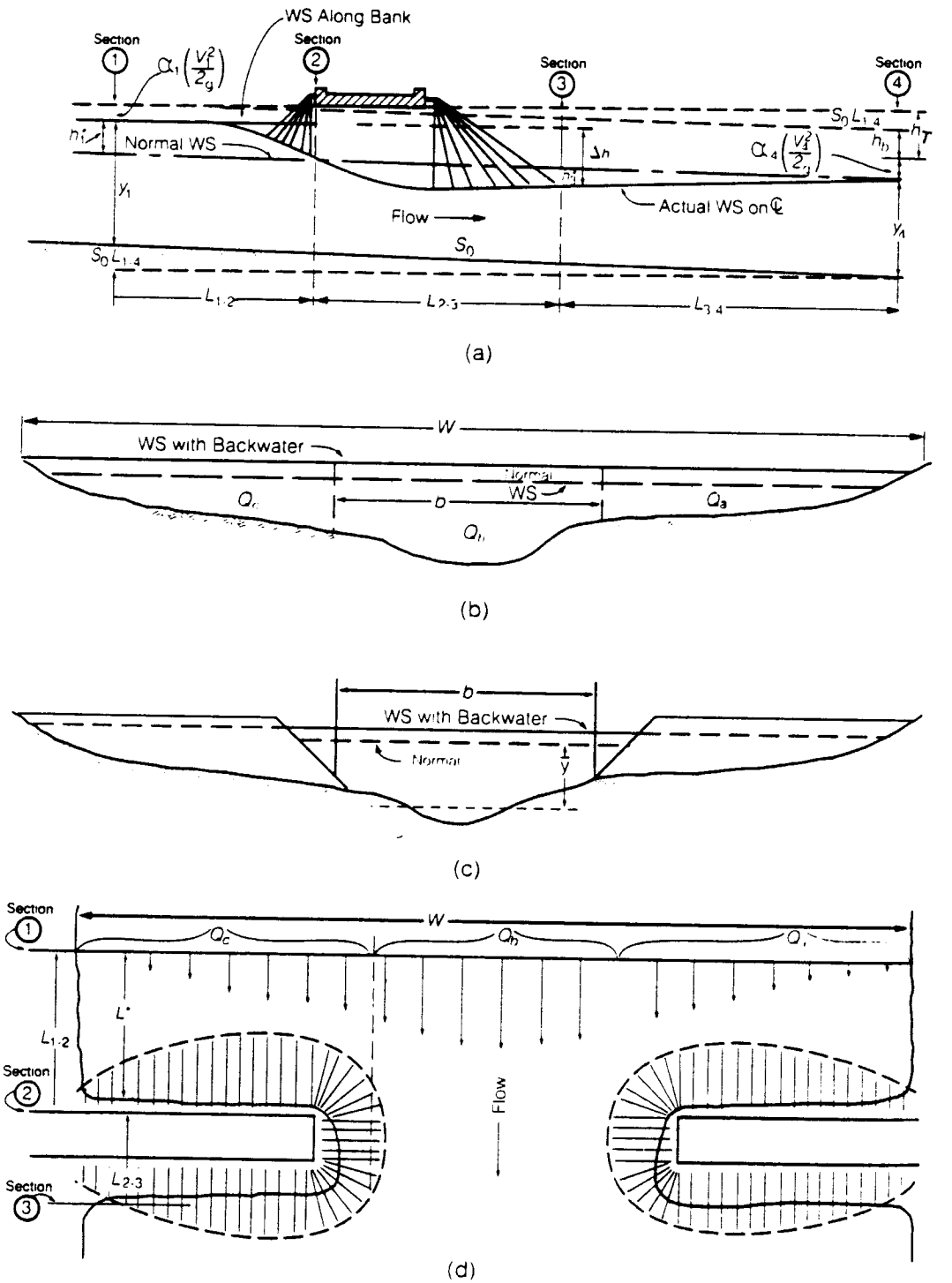


Figure 8.31—Normal crossings—spillthrough abutments: (a) profile on stream, (b) section 1, (c) Section 2, and (d) plan at bridge (U.S. Dept. of Transportation, 1970).

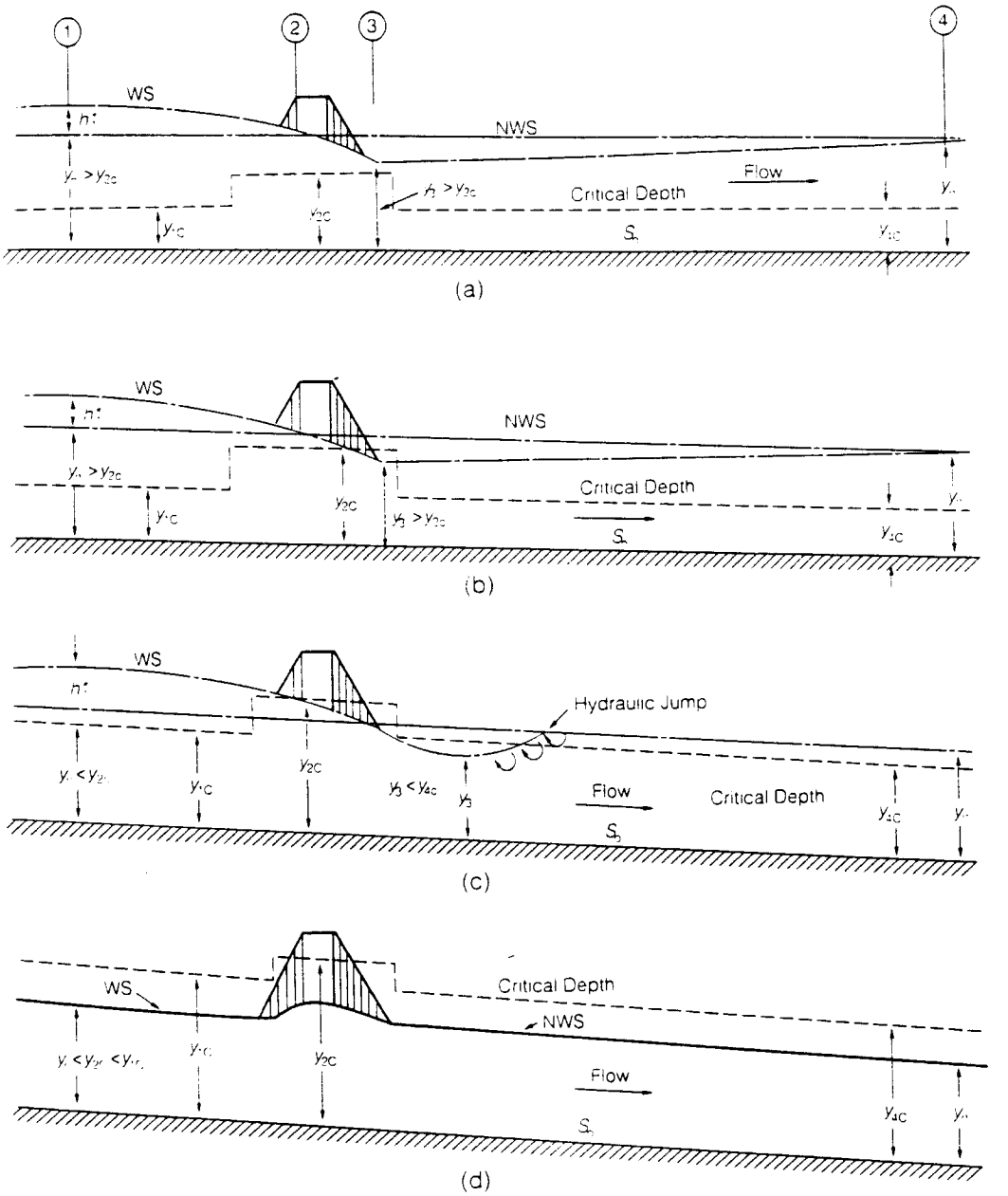


Figure 8.32—Types of flow encountered: (a) Type I Flow (subcritical), (b) Type IIA Flow (passes through critical), (c) Type IIB Flow (passes through critical), and (d) Type III Flow (supercritical) (U.S. Dept. of Transportation, 1970).

opening in essentially unhindered fashion. Important sections are labelled 1, 2, 3, and 4 (proceeding from upstream to downstream).

The manner in which flow is contracted in passing through a bridge constriction is illustrated in Figure 8.32. A very marked change is evidenced near the abutments since the momentum of the flow from both sides (or floodplains) must force the advancing central portion of the stream over to gain entry to the constriction. Upon leaving the con-

striction, the flood gradually expands until normal conditions in the stream are again reestablished.

Constriction of the flow causes a loss of energy, the greater portion occurring in the re-expansion downstream. This loss of energy is reflected in a rise in the water surface upstream of the bridge. The magnitude of this rise, or backwater, is one of the most important bridge design criteria (other important criteria include downstream channel impacts, potential scour of the bridge abutments, piers, caissons, etc., and the potential for roadway overtopping during extreme floods).

Figure 8.32 indicates the four types of flow that may be encountered at bridges. These are labelled types I, IIA, IIB and III. The long dashed lines shown on each profile represent the normal water surface, or the stage the design flow would assume prior to placing a constriction in the channel. The solid lines represent the configuration of the water surface, on centerline of channel in each case, after the bridge is in place. The short dashed lines represent critical depth, or critical stage in the main channel (Y_{1c} and Y_{4c}) and critical depth within the constriction, Y_{2c}, for the design discharge in each case.

Type I Flow Referring to Figure 8.32, it can be observed that normal water surface is everywhere above critical depth. This has been labelled type I or subcritical flow, the type usually encountered in practice. The backwater expression for type I flow is obtained by applying the conservation of energy principle between sections 1 and 4.

Type IIA Flow For type IIA flow, the normal water surface in the unconstricted channel again remains above critical depth throughout, but the water surface passes through critical depth in the constriction. Once critical depth is penetrated, the water surface upstream from the constriction, and thus the backwater, becomes independent of conditions downstream (even though the water surface returns to normal stage at section 4).

Type IIB Flow The water surface for type IIB flow starts out above both normal water surface and critical depth upstream, passes through critical depth in the constriction, next dips below critical depth downstream from the constriction and then returns to normal. The return to normal depth can be rather abrupt (hydraulic jump) since the normal water surface downstream is above critical depth. A backwater expression applicable to both types IIA and IIB flow has been developed by equating the total energy between section 1 and the point at which the water surface passes through critical stage in the constriction.

Type III Flow In type III flow, the normal water surface is everywhere below critical depth and the flow throughout is supercritical. This is an unusual case normally found only in mountainous regions. Theoretically, backwater should not occur for this flow type, since the flow throughout is supercritical. It is likely that an undulation of the water surface will occur in the vicinity of the constriction, however.

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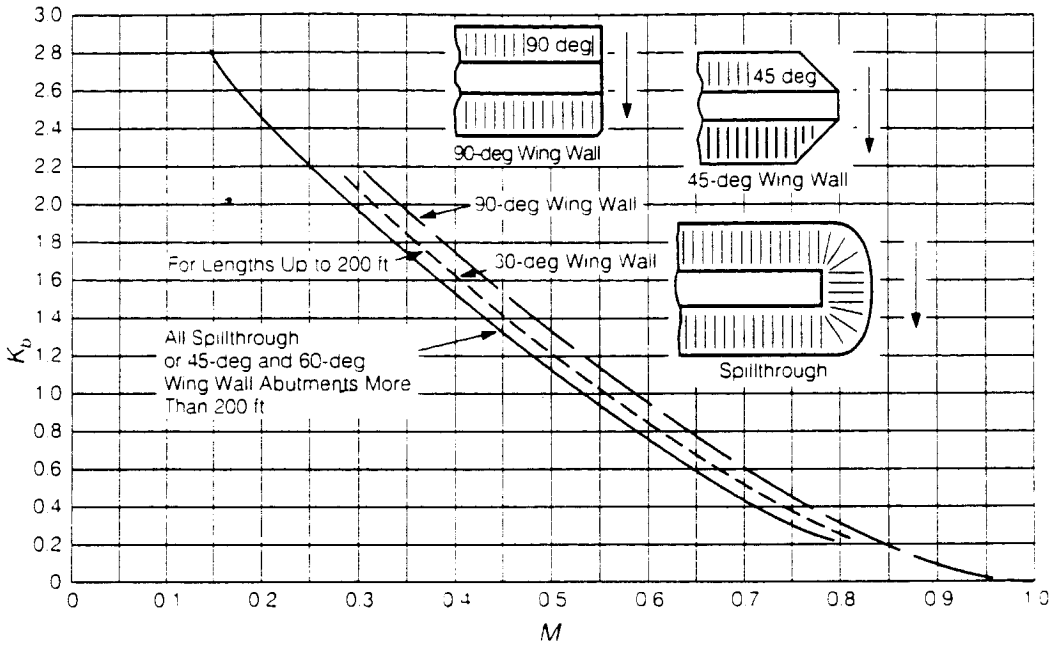

Figure 8.33—Backwater coefficient base curves (subcritical flow) (ft × 0.304 8 = m) (U.S. Dept. of Transportation, 1970).

2. Hydraulic Evaluation

A practical expression for backwater caused by bridges has been formulated by applying the principle of conservation of energy between the point of maximum backwater upstream from the bridge, section 1, and a point downstream from the bridge at which normal stage has been reestablished, section 4 (see Figure 8.33). The expression is reasonably valid if the channel in the vicinity of the bridge is essentially straight, the cross-sectional area of the stream is fairly uniform, the gradient of the bottom is approximately constant between sections 1 and 4, the flow is free to contract and expand, there is no appreciable scour of the bed in the constriction and the flow is in the subcritical range.

The expression for computation of backwater upstream from a bridge constricting the flow follows:

$$h_1^* = K^* a_2 \left(\frac{V_{n2}^2}{2g}\right) + a_1 \left(\frac{A_{n2}^2}{A_4}\right) - \frac{A_{n2}^2}{A_1}\left(\frac{V_{n2}^2}{2g}\right) \tag{8-11}$$

where

h_1^* = total backwater (feet).
K^* = total backwater coefficient. K^* is comprised of K_b (the principal loss due to the fact that a bridge opening confines flow)

and minor losses that account for piers (K_p), skew (K_s) and eccentricity (K_e).

a_1 & a_2 = as defined in U.S. Department of Transportation (1970).

A_{n2} = gross water area in constriction measured below normal stage (square feet).

V_{n2} = average velocity in constriction or Q/A_{n2} (feet/second)

A_4 = water area at section 4 where normal stage is reestablished (square feet).

A_1 = total water area at section 1, including that produced by the backwater (square feet).

To compute backwater, it is necessary to first obtain the approximate value of $h_1{}^*$ by using the first part of Equation (8-11):

$$h_1{}^* = K^* a_2 \left(\frac{V_{n2}^2}{2g} \right) \tag{8-12}$$

The value of A_1 in the second part of Equation 8-11, which depends on $h_1{}^*$, can next be determined and the second term of the expression then evaluated:

$$A_1 \left[\frac{A_{n2}^2}{A_4} - \frac{A_{n2}^2}{A_1} \right] \frac{V_{n2}^2}{2g} \tag{8-13}$$

This part of the expression represents the difference in kinetic energy between sections 4 and 1, expressed in terms of the velocity head, $V_{n2}^2/2g$.

The symbol K_b is the backwater coefficient for a bridge in which only the bridge opening ratio, M, is considered. M is defined as the degree of stream constriction involved, expressed as the ratio of the flow that can pass unimpeded through the bridge constriction to the total flow of the river. Referring to Figure 8.33,

$$M = \frac{Q_b}{Q_a + Q_b + Q_c} = \frac{Q_b}{Q_{\text{total}}} \tag{8-14}$$

K_b may be thought of as a base coefficient and the curves on Figure 8.35 are called base curves. The value of the overall backwater coefficient, K^*, is likewise dependent on the value of M but also affected by:

(a) Number, size, shape, and orientation of piers in the constriction
(b) Eccentricity of asymmetric position of bridge with respect to the channel cross-section
(c) Skew (if bridge crosses stream at other than 90° angle).

K^* consists of a base curve coefficient, K_b, to which is added incremental coefficients to account for the effect of piers, eccentricity and

skew. The value of K^* is nevertheless primarily dependent on the degree of constriction of flow at a bridge.

Since the designer will commonly use a computer program like HEC-2 to calculate the water surface profile through bridges, it is recommended that bridge behavior be carefully checked for reasonableness using hand techniques. Plotting the water surface and energy grade line profiles is always instructive.

A good understanding of the watershed enables a sound decision to be made as to debris blockage. Debris blockage may be accounted for by increasing the cross-sectional area of the bridge opening.

XI. APPLICATION OF THE RATIONAL METHOD IN DESIGN

The Rational Formula, or one of its many variants, has been used for many years for sizing storm drainage systems. For small basins, it continues to be a reasonable method, **provided that it is used properly and that results and design concepts are assessed for reasonableness** (for a detailed discussion on the application/limitations of the method, see Chapter 5).

After the minor system is preliminarily designed and checked for its interaction with the major system, reviews are made of alternates, hydrological assumptions are verified, new computations are made, and final data obtained on street grades and elevations. The engineer then should proceed with final hydraulic design of the system. To establish street carrying capacities and storm sewer requirements at a preliminary design level, a procedure similar to that shown in Figure 8.34 (adapted from Urban Drainage and Flood Control District 1984), is suggested. The procedure is for the average situation; variations may be necessary to fit actual field conditions. It should be noted that procedures like this are particularly suitable for computer spreadsheets.

(a) Column 1—Determine design point location and list. This design point should correspond to the sub-basin illustrated on the preliminary layout map.
(b) Column 2—List basins contributing runoff to this point that have not previously been analyzed.
(c) Column 3—Enter length of flow path between previous design point and design point under consideration.
(d) Column 4—Determine the inlet time for the particular design point. For the first design point of a system, the inlet time will be equal to the time of concentration (t_c). Remember that t_c is the wave travel time, and includes both overland flow time and travel time in a discrete channel. For subsequent design points, inlet time should also be tabulated to determine if it may be of greater magnitude than the accumulated time of concentration from upstream basins. If the inlet time exceeds the time of concentration from the upstream basin, and the area tributary to the inlet is of sufficient magnitude, the inlet time should be substituted for time of concentration and used for this and

STORM DRAINAGE SYSTEM PRELIMINARY DESIGN DATA

Location of Design Point	Basins	Length ft.	Inlet Time min.	Flow Time		Time of Concentration min.	Coefficient "C"	Intensity "I" in./hr.	Acre "A" acre	Direct Runoff cfs	Other Runoff cfs	Summation Runoff cfs	Street		Pipe			Street		Pipe		Remarks
				Street min.	Pipe min.								Slope %	Allowable Capacity cfs	Slope %	Size in.	Capacity cfs	Design cfs	Velocity fps	Design cfs	Velocity fps	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23

Figure 8.34—Typical form for storm drainage system preliminary design data.

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subsequent basins. In other words, at each design point in the system, the engineer should ascertain whether the total drainage area with a composite t_c or the given individual upstream basin (with a different t_c) produces the higher discharge.

(e) Column 5—Enter the appropriate flow time (wave travel time) between the previous design point and the design point under consideration. The flow time of the street should be used if a significant portion of the flow from the above basin is carried in the street.

(f) Column 6—Pipe flow time should generally be used unless there is significant carry-over from above basins in the street.

(g) Column 7—The time of concentration is the summation of the previous design point time of concentration and the intervening flow time.

(h) Column 8—Rational Method Runoff Coefficient, "C", for post-development conditions for the basins listed in Column 2, should be determined and listed. The "C" value should be weighted if the basins contain areas with different "C" values.

(i) Column 9—The intensity to be applied to the basins under consideration is obtained from the intensity-duration-frequency curve developed for the specific area under consideration based upon depth-duration-frequency information. The intensity is determined from the time of concentration and the return frequency for this particular design point.

(j) Column 10—The area in acres of the basins listed in Column 2 is tabulated here. Subtract ponding areas which do not contribute to direct runoff such as rooftop and parking lot ponding areas.

(k) Column 11—Direct runoff from the tributary basins listed in Column 2 is calculated and tabulated here by multiplying Columns 8, 9, and 10 together.

(l) Column 12—Runoff from other sources, such as controlled releases from rooftops, parking lots, base flows from groundwater, and any other source are listed here.

(m) Column 13—The total of runoff from the previous design point summation plus the incremental runoff listed in Column 11 and 12 is listed here.

(n) Column 14—The proposed street slope is listed in this column.

(o) Column 15—The allowable capacity for the street is listed in this column. Allowable capacities should be calculated in accordance with procedures set forth in the "Streets" section of this chapter.

(p) Column 16—List the proposed pipe grade.

(q) Column 17—List the required pipe size to convey the quantity of flow necessary in the pipe (round up to the next commercially available pipe size).

(r) Column 18—List the capacity of the pipe flowing full (with the slope expressed in Column 16).

(s) Column 19—Tabulate the quantity of flow to be carried in the street.

(t) Column 20—List the actual velocity of flow for the volume of runoff to be carried in the street.

(u) Column 21—List the quantity of flow determined to be carried in the pipe.

(v) Column 22—Tabulate the actual velocity of flow in the pipe for the design Q.

(w) Column 23—Include any remarks or comments that may affect or explain the design. The allowable quantity of carry-over across street

intersections, if any, should often be listed for the minor design storm. When routing the major storm through the system, required elevations for adjacent construction can often be listed in this column.

Inlet design is not specifically accounted for with Figure 8.34, although it can be by designating each inlet a "Design Point" in Column 1 of Figure 8.34. Inlets are designed using local values for *tc* (flows to individual inlets are from small areas) whereas the procedure in Figure 8.34 applies to an entire storm sewer/street conveyance system, including the total tributary drainage area.

Figure 8.34 results in a preliminary design only. Final design involves computing the hydraulic grade line for the proposed storm sewer system for the design flow, with pipe size adjustments as necessary until the desired hydraulic grade line is obtained.

XII. ECONOMIC CONSIDERATIONS

It is axiomatic in the public works profession that: (1) all works should be built for permanency, and (2) nothing should be built which cannot be maintained. Surveys (Urban Drainage and Flood Control District 1989) have shown that a weighted mean recommended project life was 58 years. Others have recommended service lives for urban storm drainage projects of between 50 and 100 years (see Chapter 14 for further discussion of design life).

The designer of stormwater management facilities focuses on two kinds of economic evaluation:

(a) Cost-effectiveness of alternatives during conceptual and preliminary design.
(b) Preparation of detailed cost estimates for the preferred alternative.

A. ECONOMIC COMPARISON OF ALTERNATIVES

The stormwater master plan for any development typically presents various alternatives from which the preferred alternative is selected. One criterion in the selection process (normally the most important) is the cost of each alternative. Costs for local drainage systems consist of capital, operation and maintenance and replacement expenses, as well as the cost of money, reflected in inflation and interest rates.

The economic evaluation of alternatives must consider frequency of flooding. If the "least cost" alternative subjects property owners to frequent flooding, increases the historic likelihood of flooding, or otherwise fails to accomplish the principal goal of the design effort—protection of lives and property—the alternative should be rejected. The level of protection that each alternative offers is an integral part of the cost of that alternative. When sufficient data are available, the local regulatory review process provides latitude with respect to design frequency selection, and when the engineer's work scope and budget are sufficiently broad, the engineer can calculate the total annual cost of all alternatives being considered.

1. *Procedures for Economic Evaluation*

One cannot assume that the alternative with the lowest capital cost is the most economically advantageous because flood damage reduction, pollution abatement, and other measurable benefits, along with operation and maintenance and replacement costs, must also be considered. For this reason, the engineer should quantify the present worth or annualized costs of the alternatives studied on the basis of capital, operation and maintenance, and replacement costs. Furthermore, cost studies should consider inflation and interest rates, material service life, and project design life. Finally, on larger projects, the engineer should consider the average annual flood losses (expressed in dollars) for the various alternatives, and adjust costs accordingly. On local projects, however, such flood damages are not normally calculated.

Cost comparisons for alternatives are often conveniently presented in matrices (or on "spreadsheets" if a microcomputer is available). These matrices compare alternative costs with respect to such factors as:

(a) Land requirements, including easements.
(b) Pipe installation requirements and pumping facilities.
(c) Open channel requirements and size of culverts/bridges.
(d) Excavation quantity.
(e) Detention Facility costs, and costs for water quality enhancement measures.
(f) Amount of de-watering required.
(g) Number of replacements of selected pipe materials in a hypothetical planning period (50–100 years).
(h) Annual operation and maintenance expenses.
(i) Special environmental or other permitting costs.
(j) Engineering and other costs for design and construction supervision.
(k) Contingencies.
(l) "Costs of construction," such as traffic delays or service disruption.

Specifically, estimates are prepared for capital, O&M and replacement costs, as follows:

2. *Capital Costs*

Capital costs typically include:

a. Land.
b. Easements (and related legal/administrative work).
c. Materials acquisition.
d. Site preparation and erosion control.
e. Facility Construction and site restoration.
f. Mapping and surveying.
g. Engineering/design fees.
h. Inspection.
i. Permit acquisition and compensatory measures.

It is not difficult to generate reliable local data for each of these elements, unless the project has unusual features. City and county governments and regional drainage authorities normally keep compre-

hensive bid price breakdowns covering items c, d, and e. City/county budget and finance offices have historical data on land and easement costs. Finally, local engineering firms can provide information on items f through i. The engineer should use all of these sources to generate price ranges for each capital cost, and then consult with the client to establish "most likely" cost estimates.

3. *Operation and Maintenance Costs*

Operation and maintenance costs should reflect both scheduled and unscheduled maintenance. For instance, the engineer knows that a grass-lined open channel will require mowing and debris removal a certain number of times per year (scheduled), but it is more difficult to quantify the replacement of riprap removed by exuberant children (unscheduled). Thought should be given to how to best estimate annual fees for the maintenance activities described, based on local considerations. Reliable O&M data are normally available from local governments.

4. *Replacement Costs*

Replacement costs for culverts, open channel linings, inlets, storm sewers and other elements of urban stormwater management systems can be reliably estimated in most cases. It is prudent to assume a project life of between 50 and 100 years for the various alternatives studied so that replacement factors are appropriately accounted for, recognizing that it is difficult to justify frequent (more than once every 40–50 years) tearing up of streets and embankments to replace deteriorated pipe.

5. *Material Service Life*

Material service life can be defined as that point in time when one of several things occur, including:

(a) Annual operation and maintenance (O&M) costs reach the amortized annual capital replacement cost. Money would be spent more efficiently by replacing the conveyance rather than continuing to maintain it at increasing levels.
(b) For storm sewers, pipe joint displacement causes debris blockage that may be sufficient to reduce capacity by as much as 20% (or more).
(c) Using a rating system of 0–5 with 0 being the original condition and 5 being a completely deteriorated invert, the service life may be defined as the point at which the condition reaches level 4 (replacement is warranted due to risks posed by excessive rust, small holes or deep pitting in the pipe invert, unusual deformation in plastic piping, or similar problems).

6. *Life Cycle Cost Analysis*

The different alternatives possible in any urban storm drainage design can have significantly different costs, which will include a number of widely-varying components in addition to the initial cost, including

durability or service life, maintenance costs, rehabilitation or replacement costs, inflation, and changes in interest rates. For this reason, it may be advisable to use a technique like Life Cycle Cost (LCC) analysis (James and Lee 1971) when performing economic evaluations of design alternatives, or to at least indicate to the project proponent that costs can be calculated in this way.

The various parameters involved in performing LCC analyses include:

(a) Economic study period, (np).
(b) Conveyance to be evaluated and associated material service life (n).
(c) Initial cost (IC).
(d) Interest rate (i).
(e) Inflation rate (I).
(f) Replacement cost (RE) at end of material service life (n).
(g) Number of material replacements (m) necessary within the economic study.
(h) Residual value, if any, of material at end of economic study period.
(i) Routine annual maintenance costs, if data are available.

The formula for the total life cycle, exclusive of maintenance is:

$$LCC = IC + RE\left[\frac{1+I}{1+i}\right]^{n} + \ldots + RE\left[\frac{1+I}{1+i}\right]^{mn} - \left[\frac{(m+1)n - np}{n}\right]RE\left[\frac{1+I}{1+i}\right]^{np} \tag{8-15}$$

IC is the initial construction cost. The second term is the present value of any future replacement costs that will be necessary within the economic study period. The last term is the present worth of any residual value.

The term $(1+I)/(1+i)$, which will generally be a value less than 1.0, is the inflation/interest factor. The rate of interest and rate of inflation are interrelated because of federal monetary and fiscal policies. Life cycle cost analyses are typically performed for interest/inflation differentials from 0–5% to test the sensitivity of the rates on the analysis.

7. *Non-quantifiable Factors*

One word of caution with respect to the economic comparison among alternatives follows. Factors that cannot readily be quantified in terms of dollars often influence the decision-making process. For instance, although an alternative might not be the most cost effective, it may preserve a historic landmark, valuable wetlands, or unusual wildlife habitat. A dollar value cannot necessarily be assigned to such factors. Nevertheless, local decision-makers and citizens may deem these factors to be of overriding importance, in which case the engineer would adopt other than the least cost alternative. To avoid selection strictly on the basis of cost, the engineer should involve a wide array of interest groups

with the analysis and make certain that the client's needs, limitations, desires and regulatory constraints are understood. Local policies and social desires normally do more to influence design (and cost) than any other factors.

B. Engineer's Estimate of Construction Cost

As the engineer moves through the design process, the level of refinement (and accuracy) of the cost estimate steadily improves. For instance, at the conceptual design level, cost estimates can be "rule of thumb" numbers that are based largely on the experience and judgment of the designer, and a 30% contingency may be appropriate. At the preliminary design level, the engineer will have sufficiently refined the design, perused representative bids on other projects, and obtained prices from suppliers and contractors to project estimates that are accurate to within ±20% of the final design estimate. For the final design estimate, the engineer thoroughly quantifies all construction aspects of the project in question including excavation quantities, special structures, length of pipe, probable areas requiring de-watering prior to construction, site mobilization expenses, etc.

It is important for the engineer to provide the client with the best estimate of project costs at each phase of the investigation so that the client can make necessary adjustments. Limitations on the accuracy of construction costs need to be impressed upon the client as reconnaissance level estimates simply may not reflect final design estimates. Cost information can be obtained from catalogs, interviews with suppliers and contractors, bid documents, and regular review of trade journal periodicals.

Caution must be exercised when extrapolating costs from one locale to another. Hydrologic and geotechnical factors change considerably over even short distances, and the designer must assure that cost estimates apply to the specific site in question.

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CHAPTER 9

SPECIAL STRUCTURES AND APPURTENANCES

I. INTRODUCTION

Stormwater conveyance systems require a variety of structures and appurtenances to control, divert and redirect flows, and to control velocities to minimize erosion and scour. This chapter provides some general principles, and presents basic guidelines for design of these structures. References are included for design procedures beyond the scope of this manual.

There are many hydraulic situations that fall outside the range of design parameters for which the structures in this chapter are intended. In such cases, a hydraulic model study may be necessary. Model studies will give the designer a much higher degree of certainty that his structure will perform as intended, particularly when conventional designs have been extended beyond their intended limits. Typical structures that are modeled include spillways, outlet-works, energy dissipators, stilling basins, drop structures, canal structures, river channels, fish ladders, and boat chutes. Details on hydraulic modeling can be found in Chapter 7 and in several references (U.S. Department of the Interior 1980; Davis and Sorensen 1969).

II. EROSION AND SCOUR

Erosion and local scour can result in channel degradation, in undermining and structural failure, or in loss of channel bed materials and damage to channel linings. Excessive suspended sediment in streams may result in undesirable environmental impacts, aesthetic problems, and burdensome maintenance costs.

Most unlined natural or man-made channels are affected by tractive forces of flowing waters and are subject to erosion. Channels are subject

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to intense local erosion or scour at obstructions, sudden changes in channel cross-section, drops, regions of changes in channel bed materials, and other similar conditions. The design of channels, conduits and any other structure that results in changes in flow regime should consider the following factors and provide measures for channel and outlet protection.

The main factors that provide favorable conditions for erosion and scour in a channel are high flow velocities, particularly at shallow depths, and soft and/or fine bed materials. Velocities are higher in steep channels, at changes in channel configuration, in smooth channels, and at higher discharges. Soil type largely determines the erosion potential of bed materials.

A. Determination of Scour Potential

1. Maximum Permissible Flow Velocities—Unlined Channel

Table 9.1 shows the maximum permissible velocities (those which do not cause scour) and the corresponding unit tractive force values published by Fortier and Scobey (1926) and converted by the U.S. Bureau of Reclamation (1952). This table was developed for straight and mature channels with relatively mild slopes and flow depths of less than 3 feet.

Permissible flow velocities were estimated for various cohesive and non-cohesive soils as shown in Figure 9.1. The correction factors for variations in flow depths that must be applied to values in Figure 9.1 are presented in Figure 9.2. Both figures are based on Hydrotechnical Construction (1936) and Chow (1959).

If newly designed channel or hydraulic structures, or additional discharges diverted to a drainage system, cause velocities that exceed the maximum permissible, design modifications or scour control structures are required.

2. Retardance and Permissible Velocities—Grassed Channels

Manning's roughness coefficient ("n") is also called the retardance coefficient. Various types of grasses have different retardance coefficients because of their density and length (see Table 9.2). There is a relationship between the retardance coefficient and the product of the mean flow velocity (V) and the hydraulic radius (R) (Chow 1964). This relationship is characteristic of vegetation and is practically independent of channel slope and shape. The "n" vs. VR relationship therefore is very useful in the design of vegetated channels which considers the various types of grasses that can be used in a particular climatic zone (see Figure 9.3).

The permissible velocity is that which will not result in significant erosion in a grassed channel for the design runoff event. Permissible velocities for different vegetative covers, channel slopes and soils conditions are shown in Table 9.3.

TABLE 9.1. Maximum Permisible Velocities Recommended by Fortier and Scobey and the Corresponding Unit-tractive-force Values Converted by the U.S. Bureau of Reclamation* (For straight channels of small slope, after aging).

Material	n	Clear Water		Water Transporting Colloidal Silts	
		V, fps	τ_0, lb/ft²	V, fps	τ_0, lb/ft²
Fine sand, colloidal	0.020	1.50	0.027	2.50	0.075
Sand loam, noncolloidal	0.020	1.75	0.037	2.50	0.075
Silt loam, noncolloidal	0.020	2.00	0.048	3.00	0.11
Alluvial silts, noncolloidal	0.020	2.00	0.048	3.50	0.15
Ordinary firm loam	0.020	2.50	0.075	3.50	0.15
Volcanic ash	0.020	2.50	0.075	3.50	0.15
Stiff clay, very colloidal	0.025	3.75	0.26	5.00	0.46
Alluvial silts, colloidal	0.025	3.75	0.26	5.00	0.46
Shales and hardpans	0.025	6.00	0.67	6.00	0.67
Fine gravel	0.020	2.50	0.075	5.00	0.32
Graded loam to cobbles when noncolloidal	0.030	3.75	0.38	5.00	0.66
Graded silts to cobbles when colloidal	0.030	4.00	0.43	5.50	0.80
Coarse gravel, noncolloidal	0.025	4.00	0.30	6.00	0.67
Cobbles and shingles	0.035	5.00	0.91	5.50	1.10

*The Fortier and Scobey values were recommended for use in 1926 by the Special Committee on Irrigation Research of the American Society of Civil Engineers.

3. *Tractive Force*

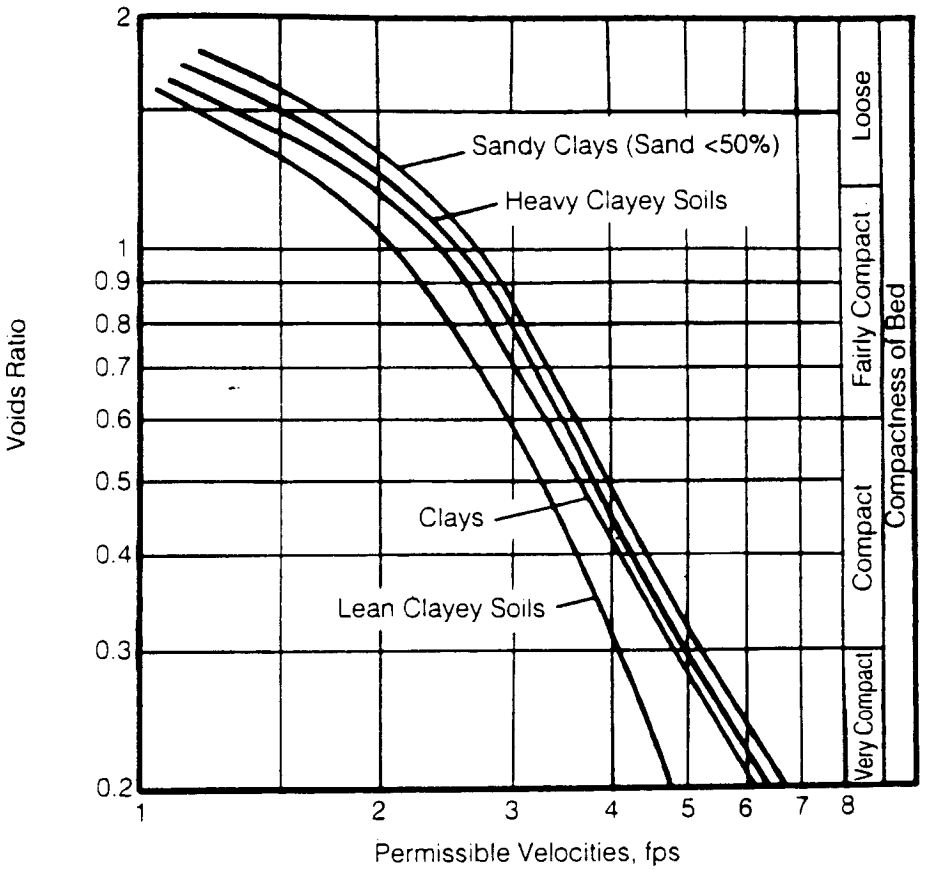
The tractive force is defined as a shear force or drag force. This force is applied on the submerged portion of the channel bed and side slopes. This force acts in the direction of flow. The unit tractive force τ, or the average value of the tractive force per unit wetted area, can be expressed (Chow 1959) as:

$$\tau = wys \tag{9-1}$$

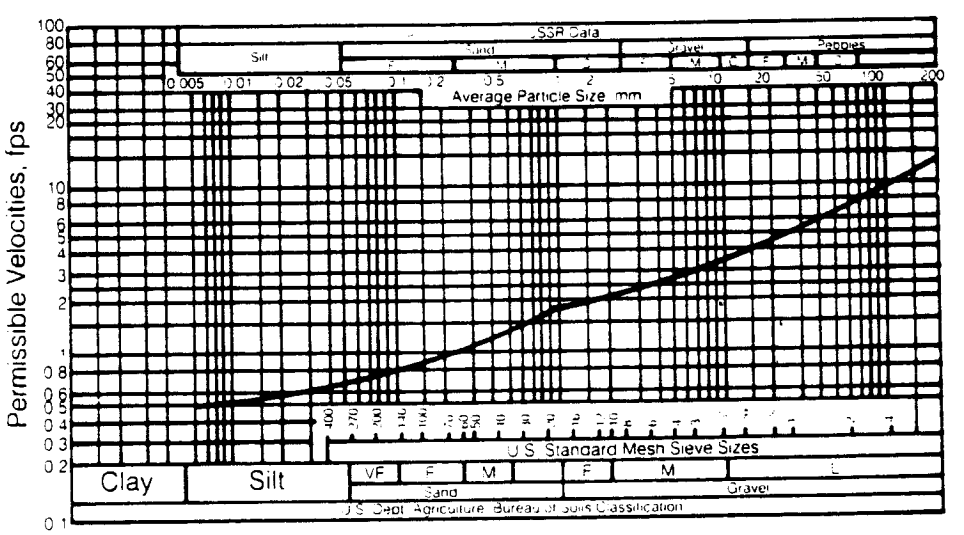
where:

w = unit weight of water
y = flow depth
S = slope

The unit tractive force in channels varies between the channel bottom and the side slopes. The maximum tractive force on the channel bottom



(a)



(b)

Figure 9.1—Curves showing data on permissible velocities for (a) cohesive soils, and (b) noncohesive soils: VF = very fine, F = fine, M = medium, C = coarse, and L = large (fps × 0.304 8 = m/s and in. × 25.4 = mm) (Hydrotech. Const., 1936, and Chow, 1959).

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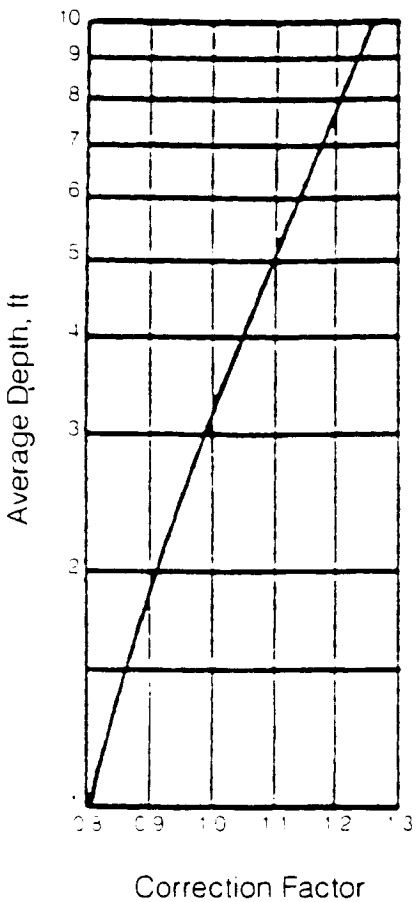


Figure 9.2—Curve showing corrections of permissible velocity for depth for both cohesive and noncohesive materials (ft × 0.304 8 = m) (Chow, 1959).

TABLE 9.2. Guide to Selection of Vegetal Retardance.

Grass Density (1)	Average Length of Grass (inches) (2)	Degree of Retardance (3)
Good	30	A Very high
	11–24	B High
	6–10	C Moderate
	2–6	D Low
	2	E Very low
Fair	30	B High
	11–24	C Moderate
	6–10	D Low
	2–6	D Low
	2	E Very low

(U.S.S.C.S. SCS-TP-61 1954)

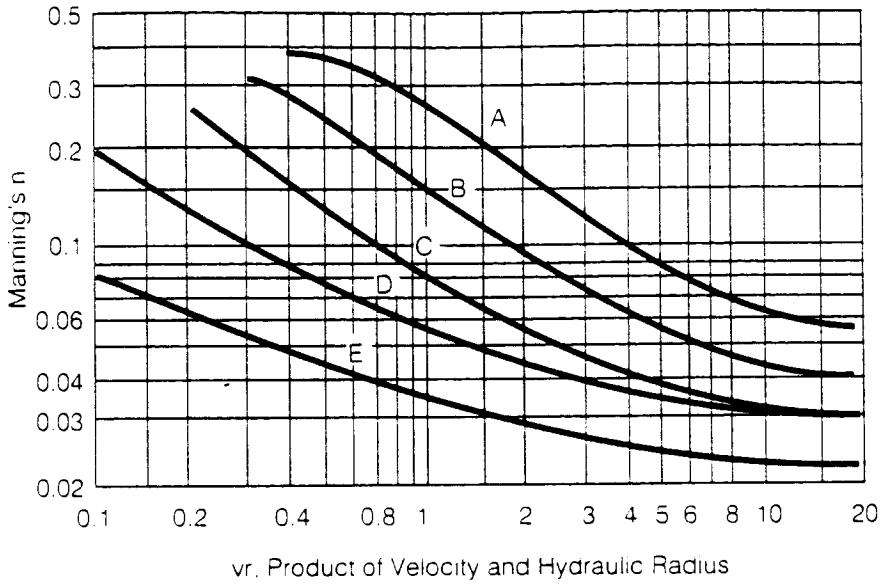


Figure 9.3—Relation between Manning's roughness coefficient and the product of velocity and hydraulic radius. The curves A *to* E *represent various degrees of vegetal retardance,* A *for very high,* B *for high,* C *for moderte,* D *for low, and* E *for very low vegetal retardance (Chow, 1964).*

is approximately *wyS*, and on the side slope is approximately 0.76 *wyS*. Figure 9.4 shows the maximum unit tractive forces on the channel bottom and side slopes (b = channel width). The maximum permissible unit tractive forces in non-cohesive materials were developed by the U.S. Bureau of Reclamation and are shown in Figure 9.5. The permissible unit tractive forces for channels in cohesive materials are presented in Figure 9.6 (Hydrotechnical Construction 1936). For further discussion of tractive force see French (1985).

B. Channel Side Slopes

From the standpoint of channel stability the minimum allowable side slopes of channels depend mainly on the soil type (As discussed in Chapter 8, many factors influence channel side slope selection including maintenance, safety, recreational use of the channel, land availability, etc.). The more cohesive the soil, the steeper the channel slope, and vice versa. Table 9.4 illustrates this concept. However, when the channel is designed in erodible materials the channel side slopes should be analyzed with the maximum permissible velocity and the principle of tractive force in mind.

C. Local Scour

Local scour occurs in non-uniform flow regions where pressure forces, lift forces, and shear forces fluctuate. For example, local scour around

TABLE 9.3. Permissible Velocities for Channels Lined with Grass

Cover	Slope Range, %	Permissible Velocity, fps: Erosion-Resistant Soils	Permissible Velocity, fps: Easily Eroded Soils
Bermuda grass	0–5	8	6
	5–10	7	5
	>10	6	4
Buffalo grass, Kentucky bluegrass, smooth brome, blue grama	0–5	7	5
	5–10	6	4
	>10	5	3
Grass mixture	0–5	5	4
	5–10	4	3
	Do not use on slopes steeper than 10%		
Lespedeza sericea, weeping love grass, ischaemum (yellow bluestem), kudzu, alfalfa, crabgrass	0–5	3.5	2.5
	Do not use on slopes steeper than 5%, except for side slopes in a combination channel		
Annuals–used on mild slopes or as temporary protection until permanent covers are established, common lespedeza, Sudan grass	0–5	3.5	2.5
	Use on slopes steeper than 5% is not recommended		

Remarks. The values apply to average, uniform stands of each type of cover. Use velocities exceeding 5 fps only where good covers and proper maintenance can be obtained.
(U.S. Soil Conservation Service 1954.)

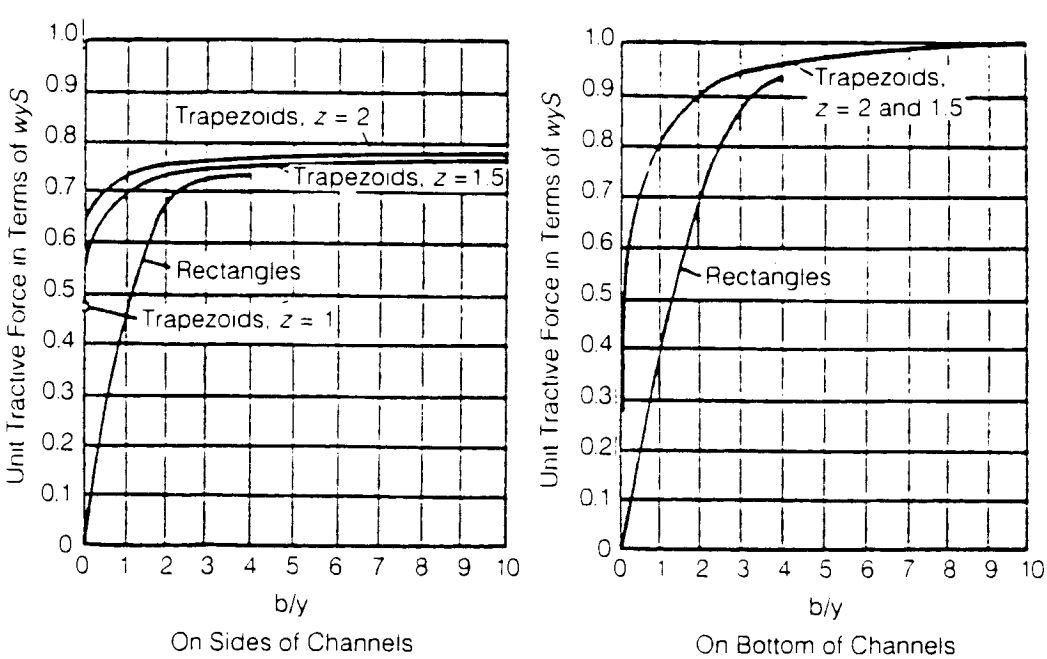


Figure 9.4—Maximum unit tractive forces in terms of wyS *(Chow, 1959).*

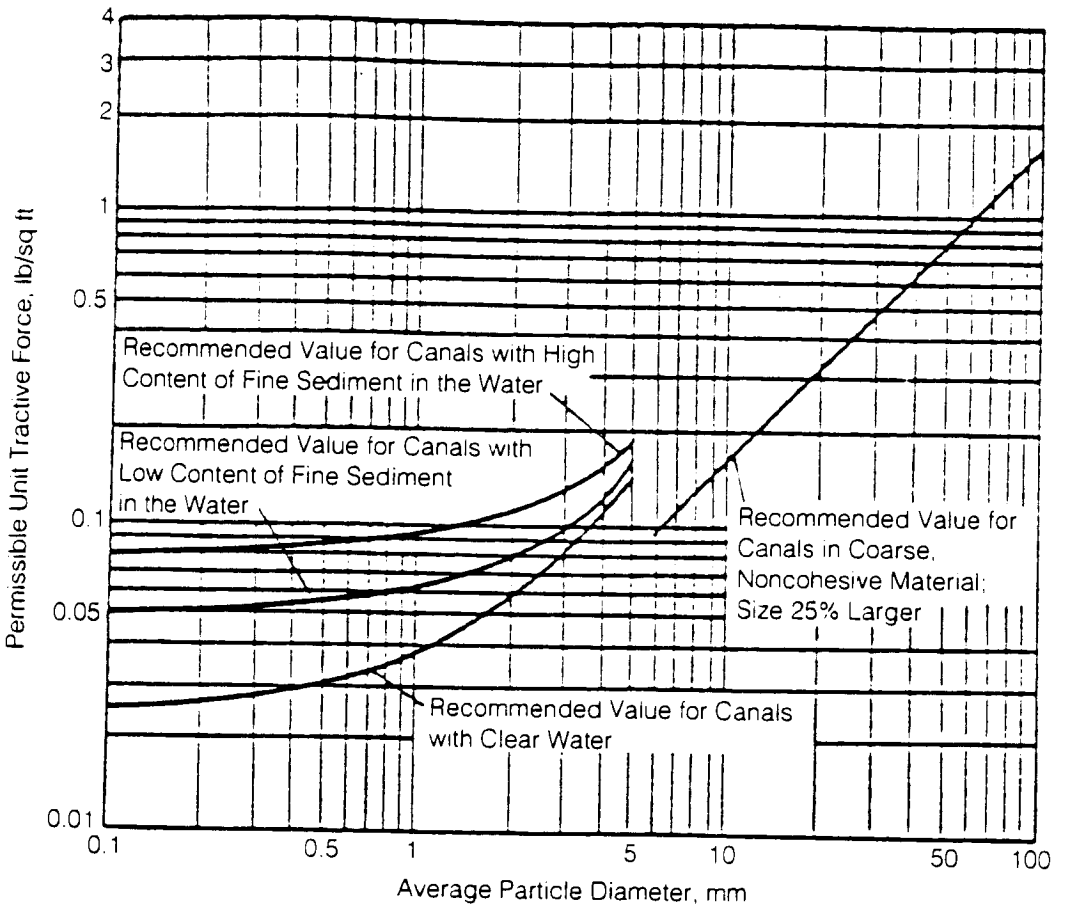


Figure 9.5—Recommended permissible unit tractive forces for canals in noncohesive material (lb/sq ft × 47.88 = Pa) (Chow, 1959).

bridge piers is caused by the vortex resulting from water piling up on the upstream edge and subsequent acceleration of flow around the nose of the pier.

Local scour is a function of a combination of several of the following factors:

(a) Slope of the channel.
(b) Characteristics of bed materials.
(c) Characteristics of the flood hydrograph.
(d) Characteristics of man-made hydraulic structures.
(e) Direction of the flow in relation to its depth.
(f) Characteristics of transported materials.
(g) Accumulation of ice and drift.

The numerous scouring factors in combination with various channel conditions and structures such as embankments, bridge piers and walls, have resulted in the development of numerous empirical relationships, based on laboratory observations, each of which applies to specific conditions. Even though the most critical factors are the flow velocity

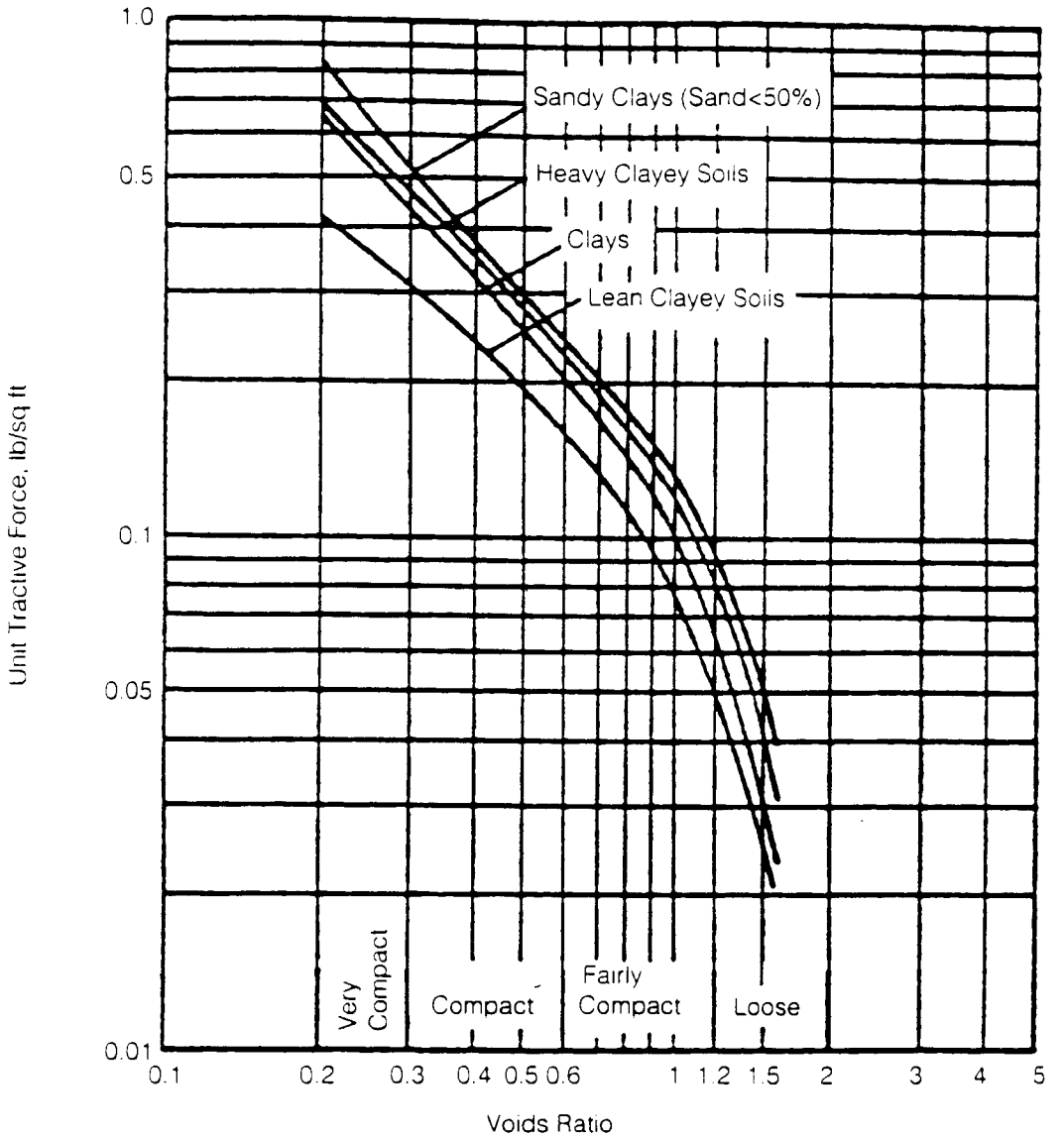


Figure 9.6—Permissible unit tractive forces for canals in cohesive material as converted from data on permissible velocities (lb/sq ft × 47.88 = Pa) (USBR and Hydrotech. Const., 1936).

and mean size and specific gravity of the sediment mixture, a comprehensive analysis should be performed for each case study using all the relevant equations and applying judgment. Most of these equations are available in ASCE (1975) and Simons and Senturk (1977). Equations and procedures for estimating scour at pipes and culvert outlets have also been published by the U.S. Federal Highway Administration (1983), and are very useful for assessing the potential scour for drainage systems including culverts and channels. These procedures are used for estimating the scour geometry based on coefficients dependent upon desired parameter such as length, width, depth, or volume of scour.

TABLE 9.4. Suitable Side Slopes for Channels (Based strictly on channel stability considerations).

Material (1)	Side Slope (horizontal to vertical) (2)
Rock	Nearly Vertical
Muck and peat soils	1/2:1
Stiff clay or earth with concrete lining	1/2:1 to 1:1
Earth with stone lining, or earth for large channel	1:1
Firm clay or earth for small ditches	1-1/2:1
Loose sandy earth	2:1
Sandy loam or porous clay	3:1

(Chow 1959).

D. Structural Measures for Channel Protection

Channel protection must be provided to suit the local physical and scour characteristics. Channel vegetation is perhaps the most simple erosion and scour control measure. Where the flow velocities exceed the velocities at which the vegetation is effective, other erosion protection measures or structures such as energy dissipators, drops, and drop shafts should be considered.

III. EROSION PROTECTION MEASURES FOR CHANNELS

A. Definitions, Categories, and Applications

Erosion protection is required for channel linings in reaches where the maximum permissible flow velocities or critical tractive forces are exceeded under the design discharge conditions.

1. Classification of Erosion Protection Measures

There are a variety of natural and man-made materials available to the engineer for erosion protection. According to "Bank and Shore Protection in California Highway Practice" (State of California 1970), bank protection systems generally can be classified as armor protection, retard protection, retaining walls, groins, and baffles. Many of these systems also can be used for bed protection. Listed below are four of the more common classifications, including examples of bank protection systems for each.

(a) Armor and Lining Protection—Armor protection is a protective surface placed to resist erosive forces. Popular armor protection includes riprap, grouted riprap, gabions, concrete, concrete rubble, sacked concrete,

shot-crete, asphalt, stone masonry, soil-cement, precast concrete revetments, and other precast materials. Lining protection can also consist of man-made and natural materials formed into mats. Several of the materials used in mat construction include concrete, plastic, geotextiles, woody plants and wood excelsior, jute, coconut, straw, and other plant fibers.

(b) Retard Protection—Retards are designed to reduce velocities and promote siltation near the toe of a bank to decrease bank erosion. Retard devices can be permeable or impermeable and include steel or concrete tetrahedrons, timber or steel jacks, fences, rock filler fences, piling, pile bents, and woody plants such as willows.

(c) Retaining Walls—Retaining walls are near-vertical structures supporting embankments or vertical side slopes in urban areas where the right-of-way is limited. Timber, concrete, masonry, and steel are all used to construct retaining walls. Retaining walls can be constructed on footings, as piling, as crib walls, and in combinations.

2. *Flexible and Rigid Erosion Protection*

The U.S. Department of Transportation (1988) has noted that the primary difference between rigid and flexible channel linings from an erosion control standpoint is their response to changing channel shape. Flexible linings are able to conform to change in channel shape while rigid linings can not. The result is that flexible linings can sustain some change in channel shape while maintaining the overall integrity of the channel lining. Rigid linings tend to fail when a portion of the lining is damaged. Damage to a lining is often from secondary forces such as frost heave or slumping. Rigid linings can be disrupted by these forces whereas flexible linings, if properly designed, will retain their erosion-control capabilities.

Flexible linings also have several other advantages compared to rigid linings. They are generally less expensive, permit infiltration/exfiltration, and have a natural appearance. Hydraulically, flow conditions in channels with flexible lining generally conform to those found in natural channels, and thus provide better habitat opportunities for local flora and fauna. In some cases, flexible linings may provide only temporary protection against erosion while allowing vegetation to be established. The vegetation will then provide permanent erosion control in the channel. The presence of vegetation in a channel can also provide a buffering effect for runoff contaminants.

Flexible linings have the disadvantage of being limited in the magnitude of erosive force they can sustain without damage to either the channel or the lining. Because of this limitation, the channel geometry (both in cross-section and profile) required for channel stability may not fit within the acquired right-of-way. A rigid channel can provide a much higher capacity and in some cases may be the only alternative.

3. *Temporary and Permanent Erosion Protection*

Erosion protection measures can also be classified as temporary or permanent. Temporary linings provide erosion protection until vegetation is established or for temporary projects usually associated with construction. In most cases the temporary protection will deteriorate over the period of one growing season which means that successful revegetation is essential. Temporary erosion protection measures include straw mats, curled wood mats, jute, paper, or synthetic nets, synthetic mats, and fiberglass roving (U.S. Department of Transportation 1988).

4. *Vegetative Erosion Protection*

Vegetative protection is suitable where uniform flow exists and shear stresses are moderate. Vegetative channel linings are not suited to sustained flow conditions or long periods of submergence. Vegetative channels with sustained low flow and intermittent high flows are often designed with a composite lining, including a riprap or concrete low-flow section (U.S. Department of Transportation 1988).

When vegetation establishes itself in a ditch line or drainage channel, then that channel is capable of handling drainage flow velocities far in excess of that handled by the soil lining alone. It is common practice to size drainage channels for vegetative linings where the two-year or ten-year rainfall event produces a design velocity of 5 fps or less and a Froude number less than 0.8. In some cases, they are analyzed first for their ability to carry a two-year storm as a soil-lined channel without vegetation, assuming that the grass will establish itself within the first two-year period after installation. After the two-year period, the ditch is assumed to be lined with a good vegetative cover and is analyzed for a ten-year or larger storm to determine its ability to handle erosive forces and the design flows under those conditions.

5. *Flow Duration*

Flow duration is a significant aspect of channel protection design. Design hydrographs should, therefore, be studied prior to design, and discharges and corresponding durations should be considered before designing a channel's erosion protection. For instance, it may not be cost-effective to riprap above a certain stage in a grass-lined channel that will only experience velocities in excess of those permissible for very short periods of time. Minor damage and the risk that goes with it may be acceptable to the owner in such cases.

6. *Permissible Shear Stress*

The permissible shear stress, p, indicates the force required to initiate movement of the protective lining. Prior to movement of the lining,

the underlying soil is relatively protected. Therefore, permissible shear stress is not significantly affected by the erodibility of the underlying soil. However, if the lining is eroded and moved, the bed material is exposed to the erosive force of the flow. The consequence of lining failure on highly erodible soils is great, since the erosion rate after failure is high compared to soils of low erodibility (U.S. Department of Transportation 1988).

Values of permissible shear stress for linings are based on research conducted at laboratory facilities and in the field. The values presented here are judged to be conservative and appropriate for design use. Table 9.5 gives permissible shear stress values for manufactured, vegetative, and riprap lining types.

B. Channel Bank and Bed Protection

1. *Riprap*

The most common and relatively inexpensive lining in most areas is riprap. Riprap is a facing or protective layer of stones randomly placed to prevent erosion, scour, or sloughing of a structure or embankment. Riprap used for slope protection includes dumped stone, hand-placed stone, wire-enclosed stone (gabion), grouted riprap, concrete riprap in bags and concrete-slab riprap. Dumped and hand-placed riprap and

TABLE 9.5. Permissible Shear Stresses for Lining Materials (U.S. Department of Transportation 1967)

Lining Category (1)	Lining Type (2)	Permissible Unit Shear Stress (lb/ft^2) (3)	Permissible Unit Shear Stress (Kg/m^2) (4)
Temporary	Woven paper net	0.15	0.73
	Jute net	0.4	2.20
	Fiberglass roving:		
	Single	0.60	2.93
	Double	0.85	4.15
	Straw with net	1.45	7.08
	Curled wood net	1.55	7.57
	Synthetic mat	2.00	9.76
Vegetative (Grass, with degree of retardance)	Class A	3.70	18.06
	Class B	2.10	10.25
	Class C	1.00	4.88
	Class D	0.60	2.93
	Class E	0.35	1.71
Gravel Riprap	1 inch	0.33	1.63
	2 inch	0.66	3.25
Rock Riprap	6 inch	2.00	9.76
	12 inch	4.00	19.53

gabions are the most common for drainage and stormwater management projects because of their relatively low initial costs and ease of replacement. While the strength of a riprap wall is somewhat limited, such a channel lining has several advantages over more impermeable linings. Riprap's porous nature protects a channel from uplift or flotation concerns, since the groundwater under pressure is able to enter the channel. In addition, riprap has the ability to conform to slight irregularities created by erosion and scour, and thus minimize additional damage.

Negative aspects of riprap linings include the difficulty of maintenance, the collection of weeds and debris, and the susceptibility to theft and vandalism in some urban areas. The economics of riprap are highly dependent upon the distance the rock must be hauled from its source to the project site.

(a) Hydraulic Considerations. Stream flow velocities, discharge, flow depth and channel bed and side slopes are the main hydraulic parameters considered in riprap design. Since riprap is subject to flow velocities and tractive forces in the channel, the size of the riprap can be estimated using tractive force criteria or design flow velocity criteria. However, in practice, the design of riprap in channels uses empirical relationships developed by various researchers and adopted by public agencies. The empirical approach, based on observations that have resulted in a great variety of charts, suggests relationships of the mean diameter or riprap to hydraulic parameters. Several references (U.S. Department of Transportation 1988; Dickinson 1968; ASCE 1975) provide comparisons of the various methods.

(b) Design Criteria. Generally, riprap may be applied to channel slopes of less than 10%. Riprap can be used on steeper slopes, but the cost may be prohibitive. Riprap selection must consider the design runoff, stone size and location, riprap gradation, thickness of riprap lining, side slopes, filter requirements and quality of stone. The design criteria are discussed below:

1. Design Runoff—The riprap is designed for the design discharge in the channel. However, it is recommended (National Academy of Science 1970) that a minimum design return interval of 25 years be used for riprap design.
2. Stone Size and Location—As indicated above there are a variety of procedures for riprap design based on empirical relationships. The latest methodology (National Academy of Science 1970) used by various states, and also recommended by the U.S. Environmental Protection Agency (1976), provides a means for estimating a design stone size (median stone diameter-d_{50}), such that the stone is stable under the design flow conditions. The procedures developed for this methodology are based on the tractive force method. Figure 9.7 shows the P/R (wetted perimeter/hydraulic radius) versus b/d (channel bottom width/flow depth) relationship (National Academy of Science 1970). The estimated P/R value is then used in Figure 9.8 to estimate the median stone diameter for a trapezoidal channel. Figure 9.9 is used for estimating the median stone diameter (d_{50}) for triangular channels.

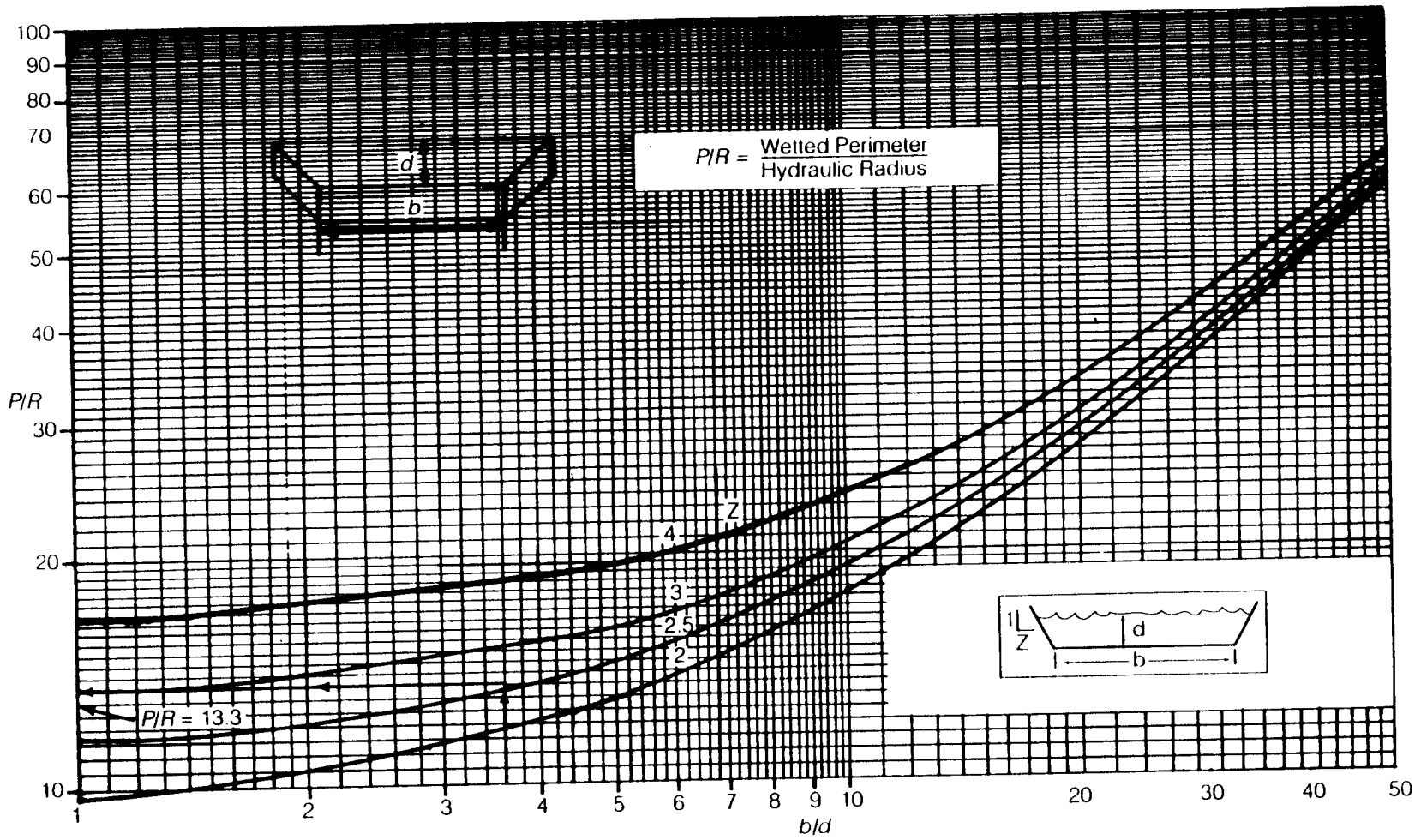


Figure 9.7—Relationship of P/R to b/d for trapezoidal channels (Highway Research Board, 1970).

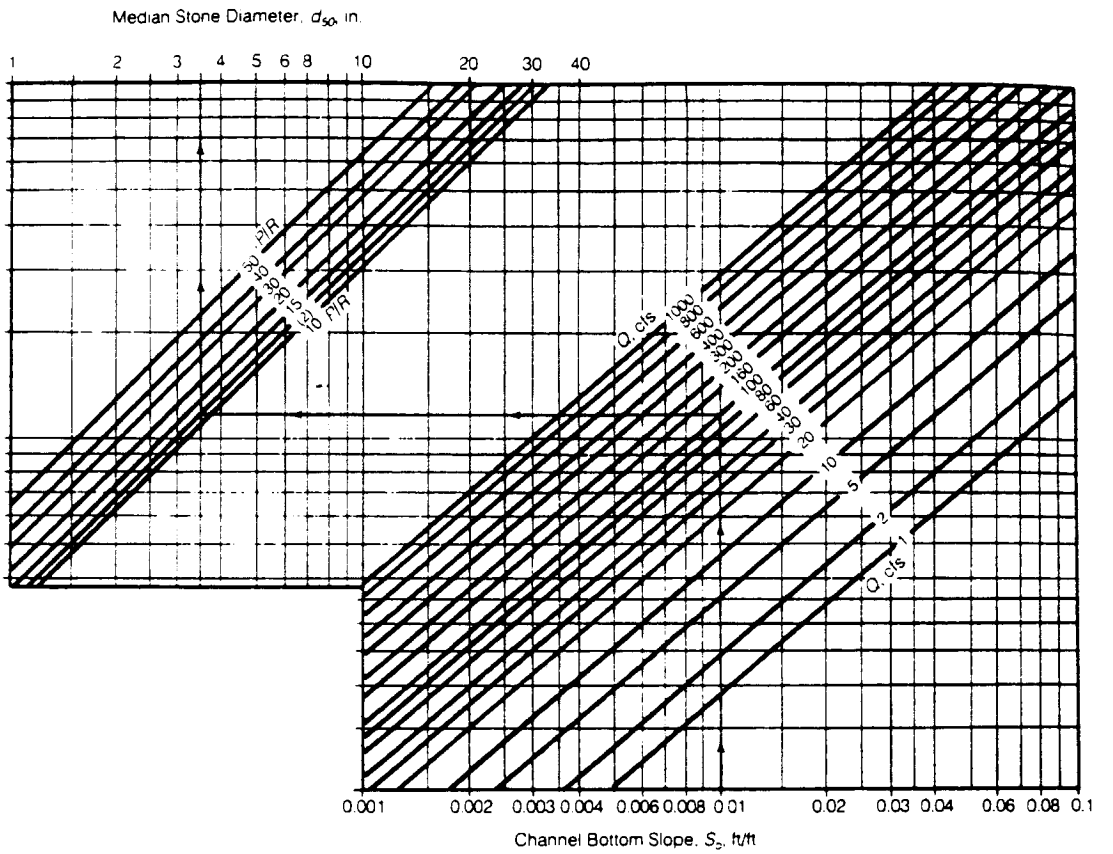


Figure 9.8—Median riprap diameter for straight trapezoidal channels: P/R = *Wetted Perimeter/Hydraulic Radius and* $d_{50} = 12(118\ QS_b^{2.2}\ R/P)^{0.4}$ *(in. × 25.4 = mm, ft/ft × 1.0 = m/m) (adapted from Highway Research Board, 1970).*

Since the erosive forces of flowing water are greater in bends than in straight channels, larger riprap must be used at the bend. Figure 9.10 can be used for increasing the d_{50} for the bend condition. It is recommended that the riprap size used in a bend extend upstream from the point of curvature and downstream from the point of tangency a distance equal to five times the channel bottom width and extend across the bottom and up both sides of the channel. As an absolute minimum, the riprap should extend to the level of the design runoff. Freeboard is recommended, however.

In channels where no riprap is required on the bottom and where the bottom consists of unconsolidated materials, the toe of the bank riprap should extend at least eight times the maximum stone size below the channel bottom. A schematic drawing for streambank riprap is shown in Figure 9.11.

3. Riprap Gradation—The riprap should be well graded and 50% of the mixture by weight should be larger than the d_{50}. The

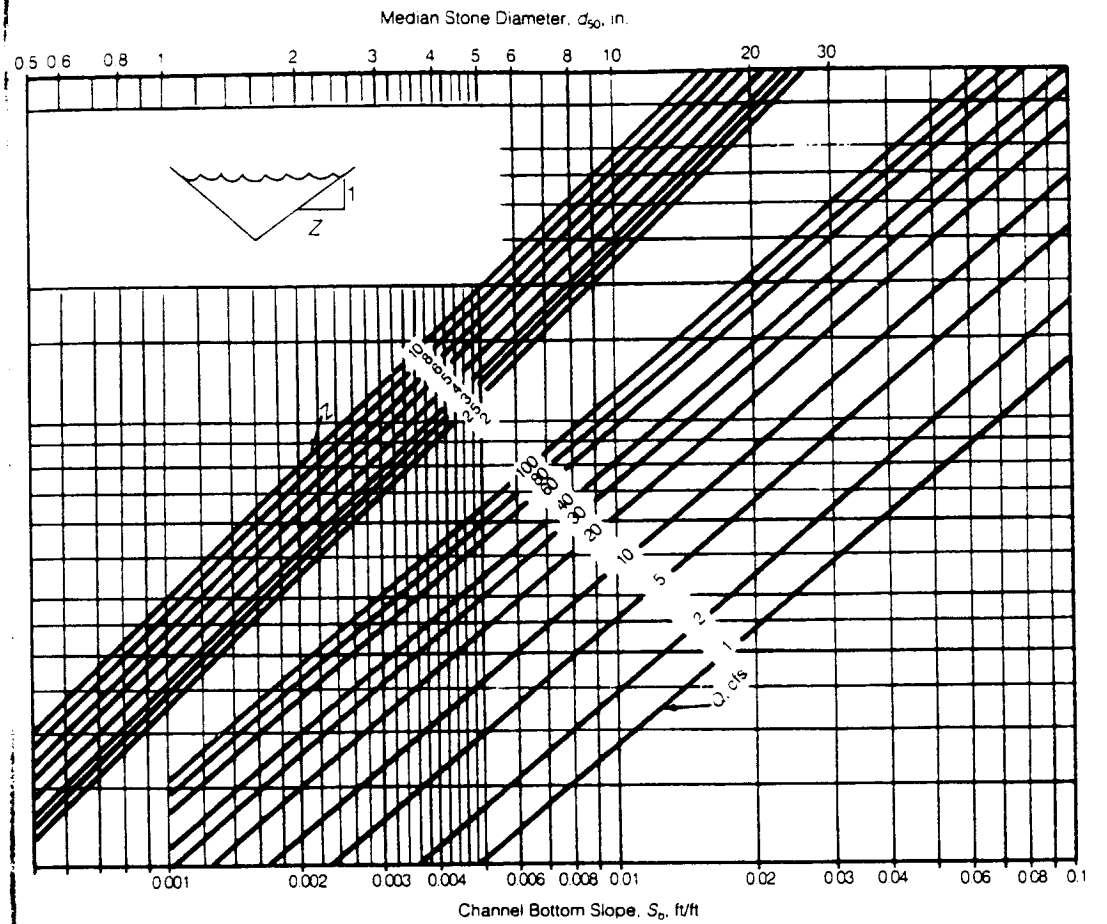


Figure 9.9—Median riprap diameter for straight triangular channels: $d_{50} = 12(64.4\ Q\ S_b^{2.2}\ Z/Z^2 + 1)^{0.4}$ *(in. × 25.4 = mm, cfs × 0.028 32 = m³/s, and ft/ft × 1.0 = m/m) (adapted from Highway Research Board, 1970).*

diameter of the largest stone should be 1.5 times the d_{50}. The minimum size of stone is that which is just stable under the design flow condition. The U.S. Department of Transportation (1967) provides riprap gradation that can be used as a guide for selection of the minimum size stone.

4. Thickness of Riprap Lining—Various parameters such as discharge, size of channel, size and gradation of riprap and construction techniques should be considered when estimating the thickness of riprap lining. The following minimum criteria should be met:
   (a) A thickness of at least three times the d_{50} if a filter layer is not used. (A filter is recommended in nearly all cases, however.)
   (b) A thickness of at least two times the d_{50} if a filter layer is used.

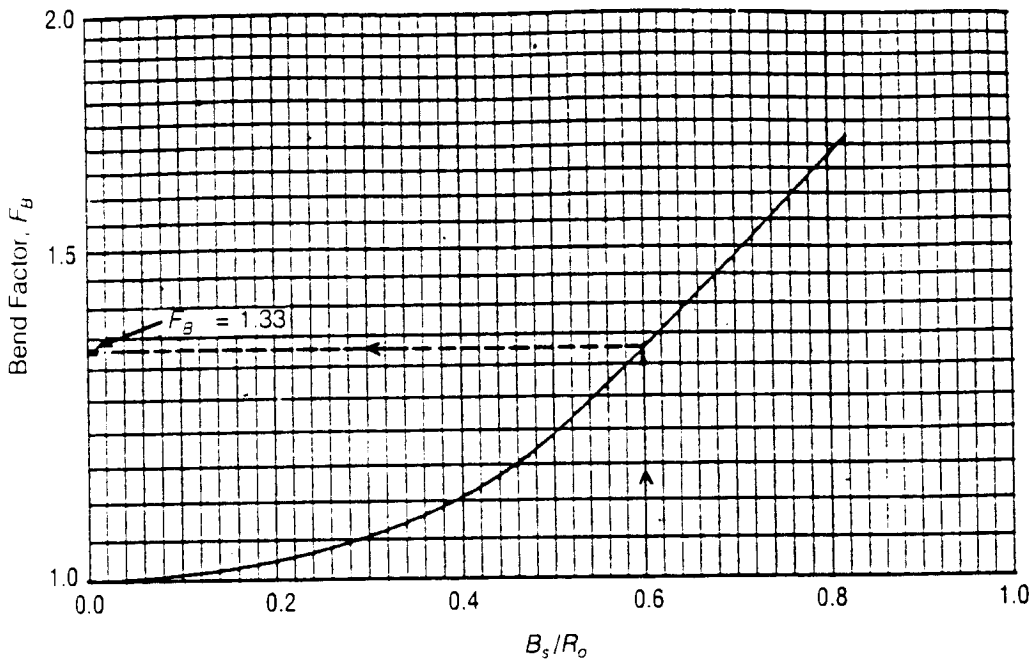


Figure 9.10—Riprap size correction factor for flow in channel bends: d_{50} *(for bend) =* d_{50} *(for straight) x* F_B, B_S *= channel surface width, and* R_O *= mean radius of bend (adapted from Highway Research Board, 1970).*

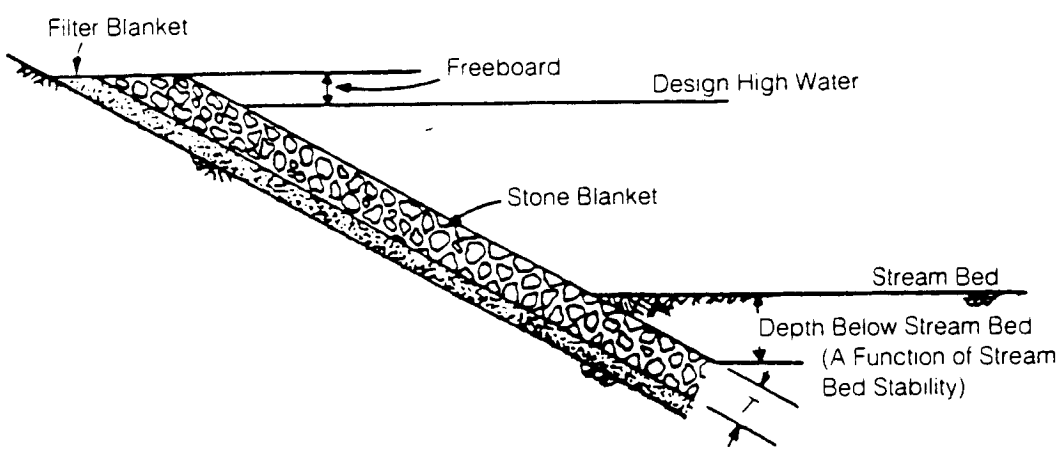


Figure 9.11—Stream bank riprap.

The minimum riprap thickness to be used is 1.0 feet.

5. Side Slopes—It is recommended that the maximum design side slopes do not exceed 2:1. Figure 9.12 shows the maximum side slopes with respect to riprap size and stone geometry. Crushed rock is more stable than rounded stone on

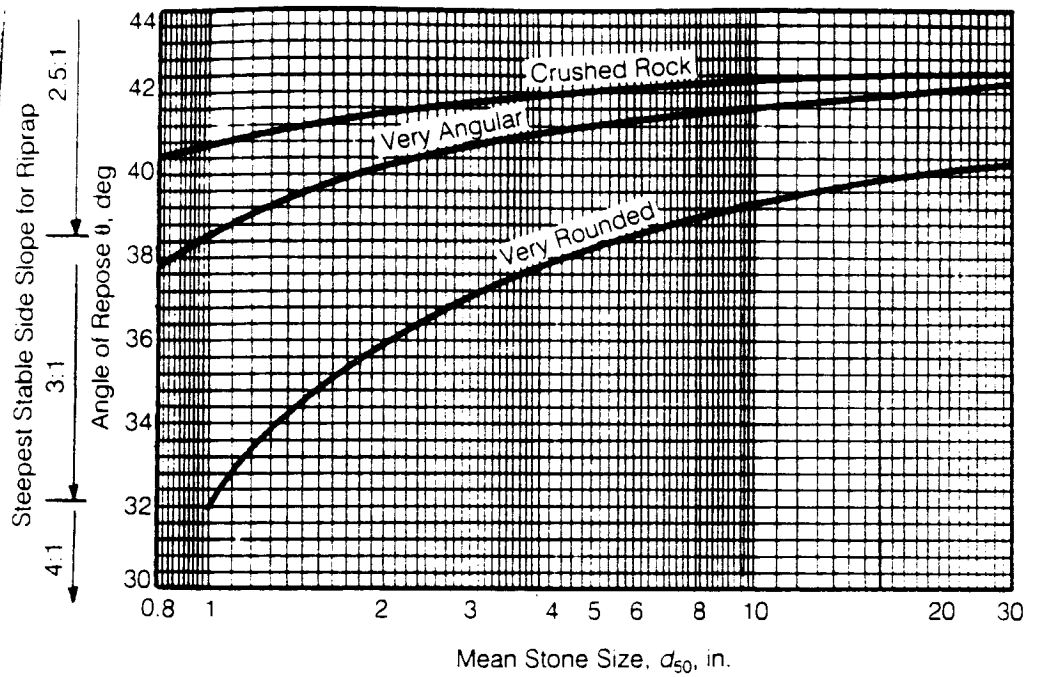


Figure 9.12—Maximum riprap side slope with respect to riprap size (in. × 25.4 = mm) (Highway Research Board, 1970).

steeper slopes. Likewise, larger stones are more stable on steeper slopes.

6. Filter—Fine soil materials underneath the riprap are subject to erosion and piping. On steep slopes, highly erodible soils, loose sand or at high flow velocities, a filter should be used or riprap thickness increased beyond the minimum referenced above.

The filter consists of either graded granular soil material that blends with the local soil or synthetic filter fabric. Criteria for detail of filters are included in a National Academy of Science (1970) publication.

Many riprap failures are caused by a lack of a filter or by use of an improperly designed filter.

7. Quality—Stone for riprap should consist of field or quarry stone of approximately rectangular shape. The stone must be hard and angular and able to withstand damage due to weathering. The specific gravity of the stone must be at least 2.5. Further discussion of riprap quality is provided in Chapter 13.
8. Installation. The proper installation of riprap and filter is critical and its importance is often overlooked. During installation, careful monitoring of riprap and filter size, gradation and thickness and assuring that the specifications are

met is important. Further discussion of riprap installation is provided in Chapter 16.

2. *Gabions*

Gabions consist of multi-celled rectangular wire mesh boxes filled with stones. Gabions are provided folded flat and bundled together for ease of shipment and handling. Each gabion is placed in position and wired to adjoining gabions, filled with rock at one third increments of height and wire cross-braced for strength and stability. After filling, the top is folded and wired shut. They have been used extensively in Europe on drainage projects of all sizes, and their use is gaining acceptance in the United States. They can be used at locations where the only rock economically available is too small for rip-rap, or where steeper side slopes are required. Another advantage is the versatility that results from the regular geometric shapes of wire-enclosed rock. The rectangular blocks and mats can be fashioned into almost any shape that can be formed with concrete.

The durability of wire-enclosed rock is generally limited by the service life of the galvanized binding wire which, under normal conditions, is considered to be about 15 years. Water carrying silt, sand, or gravel can reduce the service life of the wire, and water that rolls, or otherwise moves cobbles and large stones, breaks the wire with a hammer-and-anvil action and considerably shortens the life of the wire. The wire has been found to be susceptible to corrosion by various chemical agents and is particularly affected by high sulfate soils. If corrosive agents are known to be in the water or soil, a plastic coated wire should be specified.

The following criteria should be met when gabions are designed:

(a) The design flow velocity should not exceed the criteria indicated in Table 9.6. Table 9.6 values were taken from the New Jersey State Soil Conservation Committee (1987), and are considered minimum thicknesses permissible for the stated velocities.
(b) The gabions should not be exposed to abrasion from larger debris and/or bedload transported by the flow. The abrasion can damage the wire mesh and lead to failure.
(c) All gabions should be underlain by filter fabric or a gravel filter. Limitations on the applicability of filter fabric are provided in Table 9.7. If

TABLE 9.6. Gabion Thickness and Maximum Velocity.

Gabion Thickness (ft.) (1)	Maximum Velocity (ft./sec.) (2)
1/2	6
3/4	11
1	14

TABLE 9.7. Maximum Allowable Channel Bottom Slopes Using Geo-textile Fabrics (New Jersey State Soil Conservation Committee 1987)

Soil Texture (1)	Maximum Permissible Velocity (fps) (2)	Allowable Invert Slope (ft/ft) (3)
Sandy loam	2.5	0.029
Silt loam	3.0	0.041
Sandy clay loam	3.5	0.056
Clay loam	4.0	0.074
Clay, fine gravel	5.0	0.115
Cobbles	5.5	0.139

bottom slopes are steeper or velocities greater than those shown in Table 9.7, the gabions should be underlain with a properly designed gravel filter.

(d) Rock used to fill the gabion should be angular, block-shaped rock. Minimum rock sizes are generally 3" to 4" or ⅓ the basket depth. The maximum stone size should not exceed ⅔ the basket depth or 12", whichever is smaller.

Gabions are not maintenance-free and must be periodically inspected to determine whether the wire is sound. If breaks are found while they are still relatively small, they may be patched by weaving new strands of wire into the wire cage. Wire-enclosed rock installations need to be inspected at least once a year under ideal circumstances and may require inspection every three months in vandalism prone areas (in conjunction with a regular maintenance program). Mattresses on sloping surfaces must be securely anchored to the surface of the soil.

Where aggregate is readily available at reasonable cost, gabions can be a very economical solution to channel lining. A major factor is the cost of labor, since much of the placement of the stone, and the work in general, is very labor-intensive.

Gabions are very useful for small localized drainage problems where immediate lining solutions are needed, and which must be accomplished by local and inexperienced workmen. By installing a grout over the finished mattress installation, the effect of a paved ditch can also be gained, with possible savings in initial construction costs.

3. *Man-made Protection Materials*

In addition to riprap and gabions, several man-made erosion protection products are available. They may have advantages over riprap and gabions in specific situations, although the performance of the products is not always well documented. The designer is, therefore, advised to proceed with caution when specifying these products. It is suggested

that trial installations or other projects using these materials be evaluated.

The advantages of these materials may include consistent quality and dimensions, ease of installation, ease of underwater installation, desirable aesthetics, ease of maintenance, ease of quantity estimation, and low cost. Disadvantages may include difficulty in estimating product life, high maintenance and/or replacement costs, ease of removal by vandals, and comparatively high initial cost.

Maintenance of man-made products is often easier than that of riprap because they present an even surface to drive or walk over while mowing or conducting other maintenance activities. They should also be considered in locations where rodent infestation could be a problem in riprap voids.

A number of currently available products are described below. Case-by-case evaluation of each, coupled with performance evaluation and regular maintenance, is essential.

(a) Concrete Filled Fabric Mats—The mats are constructed of heavy-duty fabric filled with concrete. The flexible double-walled fabric framework is pressure injected with fine grain concrete. The fabric forms are available in a number of sizes, with or without filter points, and in thicknesses ranging from 3–12 inches. The mats can be installed with low headroom, underwater, and with relative speed. Several of the mats are available with reinforcing cords which provide tensile strength.

(b) Fabric Soil Stabilization Mats—The mats are a three-dimensional product made from heavy fabric monofilaments fused at their intersections. The bulky open construction allows for backfilling with soil, gravel, or other appropriate materials. Vegetation is then planted within the mat which acts as permanent turf reinforcement. The mats are generally available in thicknesses up to 0.75".

(c) Precast Concrete Revetments—The revetments are available in various shapes and sizes from a number of suppliers. Many are designed to allow vegetation to grow up in void areas. Several have interlocking features or are secured with mats or cords which reduce vandalism and potential removal by high velocity flows.

(d) Fabric Grids—Fabric grids are manufactured as a three-dimensional, semi-rigid matrix. The honeycomb-type designs are made in sizes up to 4 inches deep, with hexagon shapes having side dimensions of up to 8 inches. After the grids are unrolled on a surface, they are filled with native soils, gravel, cobble, or mixtures of soil and rock. In submerged applications, gravel or cobble is preferred, while the use of native soils that can support vegetation is an option above normal water surface elevations. Rocks can be mixed with the soil for increased erosion protection. The fabric grid adds strength and erosion resistance to the material placed in it.

The grids are anchored to the slope to protect against movement. According to one manufacturer, the grids are inexpensive, easy to install, permeable, lightweight, and rot-proof.

4. *Common Channel Protection Measures Other Than Riprap and Gabions*

(a) Bagged Concrete—In this technique, bags filled three-quarters full with concrete are laid in close contact, with staggered joints and tied ends turned in. The consistency of the concrete must be as stiff as satisfactory discharge from the mixer and the bagging process permit. Bagged concrete may be used when all the following conditions are met:
1. The design runoff, riprap size and location, and filter criteria are met.
2. The weight of the filled bags is at least equal to the weight of the maximum stone size required for rock riprap.
3. Settlement or lateral movement of foundation soils is not anticipated.
4. Ice conditions are not severe.
5. Slopes should be 2:1 or flatter. However slightly steeper slopes may be permitted under special circumstances.

(b) Soil-Cement—Soil cement is a concrete product formed by the mixing of on-site soils with portland cement in a "pug-mill" erected at the project site. The resulting mixture is a low-slump soil cement concrete with a low compressive strength, frequently between 600 and 900 pounds per square inch. A typical mixture contains about 6–12% cement and 8–12% moisture by weight. Soil-cement requires a less costly aggregate than concrete because more fines are acceptable. Soil-cement is normally placed and compacted in 6–8-inch lifts in a stair-stepped fashion (or on a single level on gentle slopes). Its primary advantages are low cost, durability, and low permeability. Soil cement may be used along with wire mattresses and other industrial fabrics to provide a stable ditch lining. Factors which tend to erode soil cement linings include rapidly changing water depths as well as freezing and thawing conditions.

Soil cement has been employed in the construction of levees, channel bank protection, and drop structures, and merits consideration as a substitute for riprap protection in areas where rock is not economically available, or where flow velocities would be too high for riprap linings.

(c) Soil-Bioengineering—Soil-bioengineering is a bank protection method which involves the use of live woody plant material such as willows. The plant materials are used in various systems and configurations to meet specific project requirements. The native plant material is designed to root and grow into structures including live crib walls, live stakes, joint planting, live soft gabions, brush-layering, brush-mattresses, and live fascines. The plant material can be used by itself or in conjunction with other bank protection materials. These systems provide the most "natural" form of bank protection and may offer low initial costs, but do require specialized maintenance. The level of maintenance is dependent on the type of system, the local climate, and desired appearance.

Reference materials that are useful in preparing site-specific designs are limited. A number of publications are available from the U.S. Army Corps of Engineers, U.S. Fish and Wildlife Service, U.S. Forest Service, and the U.S. Environmental Protection Agency. District Soil Conservation Service

offices are another source. Private publications from practicing professional hydrologists and wetland biologists are a good source of applied information. Specialists trained in bioengineering have recently begun working in the field of urban storm drainage, and their projects and publications will also be of interest.

(d) Retaining Walls. Various types of traditional retaining walls are used in special situations where the higher cost is warranted. Types of walls include crib walls, sheet pile walls, timber pile walls, concrete walls, and boulder walls. Special precautions such as filter layers are advised to guard against loss of native soils through the walls.

(e) Concrete. Concrete is by far the most widely used channel lining where grass or soil are not sufficient, especially when high Froude numbers and/or velocities indicate the need for a very reliable structure. Although more expensive than other alternatives, concrete has distinct advantages such as low maintenance, ease of installation, excellent durability, and resistance to high velocities. Architectural concrete finishes are available to improve appearance.

Most conditions allow for poured in-place concrete, although precast panels have been successfully used in channel lining applications. Concrete can be used to line all channel shapes from rectangular to trapezoidal and "V" ditch sections. A typical section of structural concrete lining may be a trapezoidal section using 6" thicknesses of concrete with welded wire fabric reinforcement laid over stone or some other porous material. Expansion joints must be provided at routine intervals as well as control joints for crack control. Weep holes are routinely installed at 6–10 foot intervals allowing the release of water which has accumulated around the outside of the concrete surface. This reduces the effects of hydrostatic uplift as well as the effects of freezing and thawing which can cause deterioration of the lining. Low strength concrete (around 2500 psi) is often used for these purposes. The finish is a rough broom finish. While side slopes as steep as 1:1 are possible, they are normally 2:1 or flatter.

A concrete channel lining provides for a reduced resistance to flow (a Manning's "n" smaller than most other lining materials), which can allow the channel to carry significantly more flow and allow for flatter slopes. An important engineering consideration is the matter of safety, since such channels are often specified in urban or urban transition areas that allow direct access to the drainage ways by children and others. The characteristic high velocities and turbulence created by drainage flow in these channels creates a dangerous situation which must be taken into account by using fences or other barricade-type devices.

(f) Roller compacted concrete (RCC) is similar to soil cement except that a mixture of sand and coarse aggregate replaces the on-site soils. RCC is placed in the same manner as soil cement and has become popular

for use in massive concrete structures such as dams. The use of RCC eliminates or greatly reduces the need for concrete forming.

(g) Bituminous Mixtures. Bituminous materials offer an economical and easily installed method of lining channels. Unfortunately, there are many problems with the use of this material that cannot be overlooked by the engineer. Oils in the lining material tend to be leached as the material degenerates, causing environmental problems. Bituminous materials, being flexible pavement, are generally kept "alive" by use. This quality makes bituminous materials excellent for road surfacing, but when they are used in a ditch without such repetitive load conditions, bituminous materials have a tendency to degenerate. In areas subject to freezing and thawing, the bituminous surface should be kept sealed to prevent the degenerative action of water and ice. Vegetation may also root through cracks causing crumbling of the surface. Bituminous paving of channels for the above reasons can only be expected to produce a serviceable channel lining for a limited life span. Where channel aesthetics are not a factor and if a limited life span is desired, bituminous linings should be considered. This solution then represents an easily installed low cost lining of a temporary nature.

(h) Corrugated Steel Sheet Channel Liner. Corrugated steel sheets may offer an economical means of lining open channels or flumes. Corrugated steel sheets can be fabricated in many sizes and shapes with a wide range of protective coatings designed to resist soil and water conditions. Fittings include tees, wyes, laterals and elbows fabricated from the same material. The open curved sheets are made up on standard lengths of 25–1/2" which are lapped, resulting in usable lengths of even 2' multiples. The sections are connected at the lap by field bolting through pre-punched holes. Flanges are provided at the longitudinal edges for anchorage.

C. Riprap Protection at Outlets

If the flow velocity at a conduit outlet exceeds the maximum permissible velocity for the local soil or channel lining, channel protection is required. This protection usually consists of an erosion resistant reach, such as riprap, between the outlet and the stable downstream channel to provide a stable reach at the outlet in which the exit velocity is reduced to a velocity allowable in the downstream channel. The design of such protection is normally based on a 25-year design runoff event.

If protection is needed at the outlet, a horizontal (zero slope) apron must be provided.

1. Apron Dimensions

(a) The length of an apron (L_a) is determined using the following empirical relationships that were developed for the U. S. Environmental Protection Agency (1976):

$$L_a = \frac{1.8Q}{D_0^{3/2}} + 7D_0, \text{ for } TW < \frac{D_0}{2} \qquad (9\text{-}2a)$$

and

$$L_a = \frac{3Q}{D_0^{3/2}} + 7D_0, \text{ for } TW > \frac{D_0}{2} \tag{9-3a}$$

where:

D_o = maximum inside culvert width (ft)
Q = pipe discharge (cfs)
TW = tailwater depth (ft)

In metric units, these equations become:

$$L_a = \frac{3.26Q}{D_0^{3/2}} + 7D_0 \tag{9-2b}$$

and

$$L_a = \frac{5.44Q}{D_0^{3/2}} + 7D_0 \tag{9-3b}$$

respectively.

(b) Where there is no well defined channel downstream of the apron, the width, W, of the outlet and of the apron (as shown in Figure 9.13) should be as follows:

$$W = 3D_0 + 0.4L_a, \text{ for } TW \geq \frac{D_0}{2} \tag{9-4}$$

and

$$W = 3D_0 + L_a, \text{ for } TW < \frac{D_0}{2} \tag{9-5}$$

The width of the apron at the culvert outlet should be at least 3 times the culvert width.

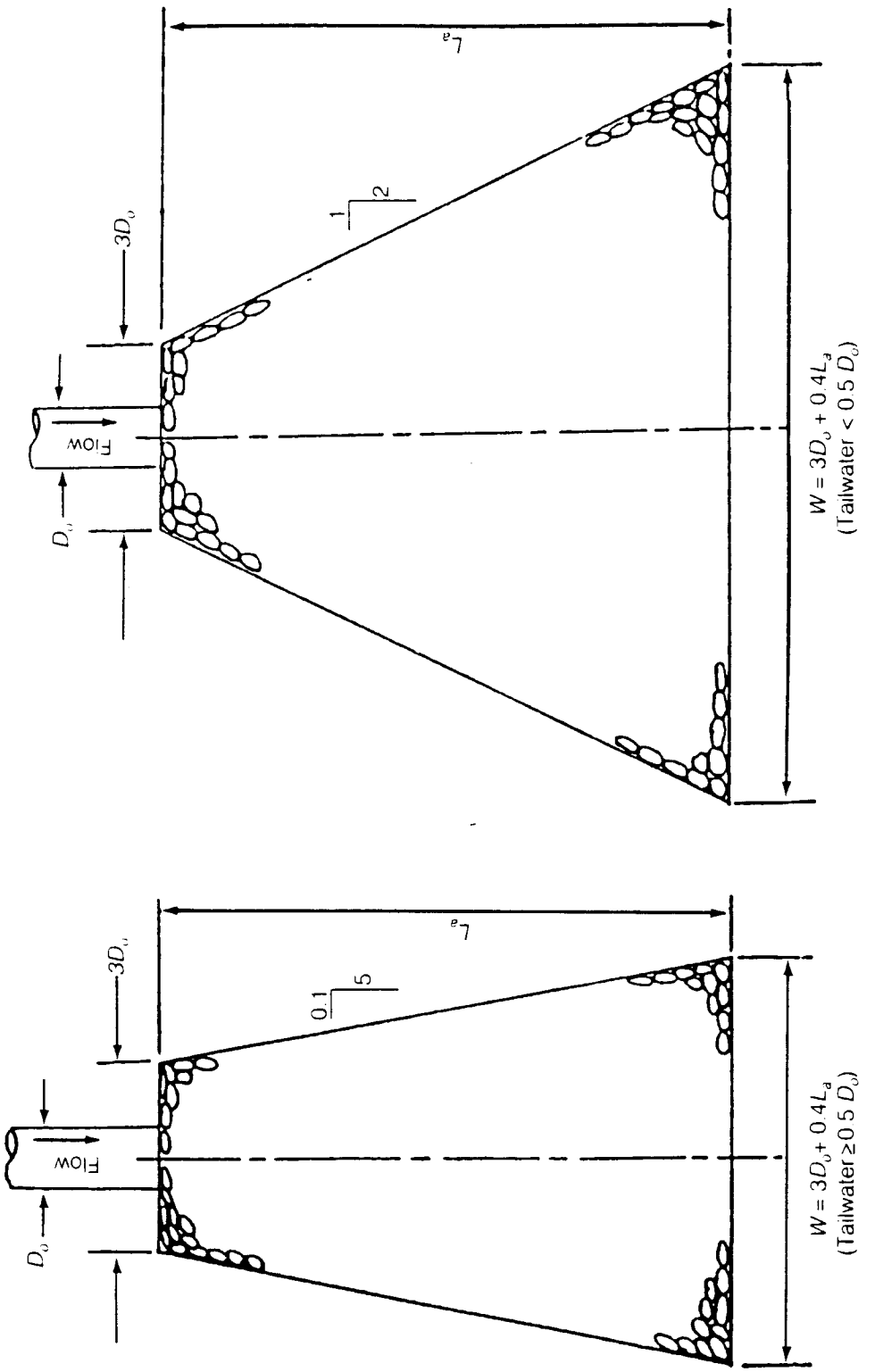


Figure 9.13—Configuration of conduit outlet protection (U.S. EPA, 1976).

(c) Where there is a well-defined channel downstream of the apron, the bottom width of the apron should be at least equal to the bottom width of the channel and the lining should extend at least one foot above the tailwater elevation and at least two-thirds of the vertical conduit dimension above the invert.
(d) The side slopes should be 2:1 or flatter.
(e) The bottom grade should be level.
(f) There should be overfall at the end of the apron or culvert.

2. Apron Materials

(a) The median stone diameter, d_{50} is determined from the following equation:

$$d_{50} = \frac{0.02\ (Q)^{4/3}}{TW\ (D_0)} \tag{9-6a}$$

and, in metric units:

$$d_{50} = \frac{0.066\ (Q)^{4/3}}{TW\ (D_0)} \tag{9-6b}$$

(b) Existing scour holes may be used where flat aprons are impractical. Figure (9.14) shows a general design of a scour hole. The stone diameter is determined using the following equations:

$$d_{50} = \frac{0.0125\ (Q)^{4/3}}{TW\ (D_0)}, \text{ for } Y = \frac{D_0}{2} \tag{9-7a}$$

and, in metric units:

$$d_{50} = \frac{0.041\ (Q)^{4/3}}{TW\ (D_0)} \tag{9-7b}$$

Also,

$$d_{50} = \frac{0.0082\ (Q)^{4/3}}{TW\ (D_0)}, \text{ for } Y = D_0 \tag{9-8a}$$

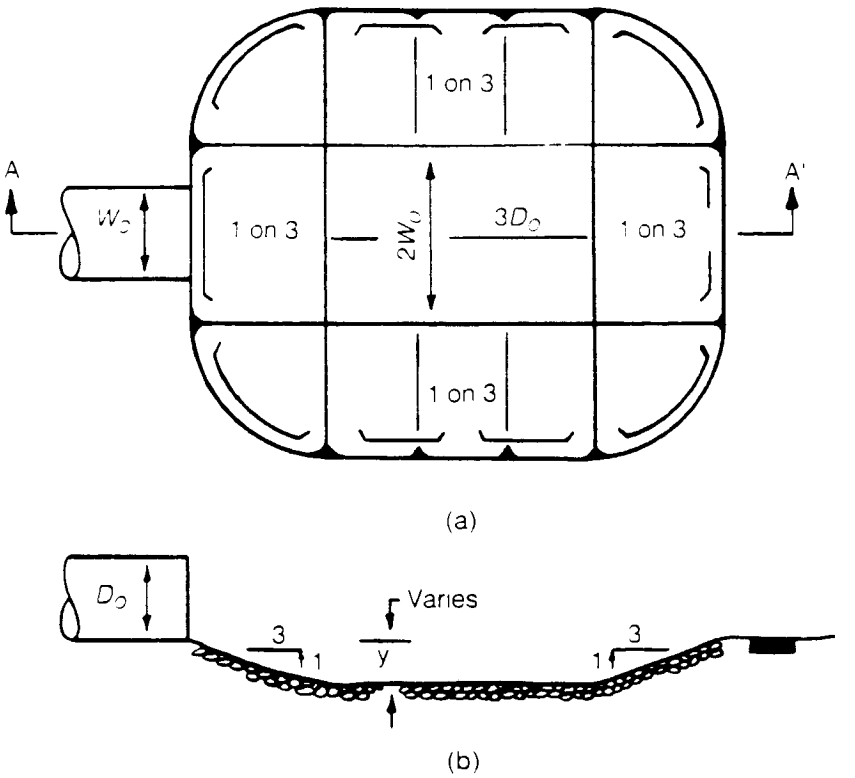


Figure 9.14—Preformed scour hole: (a) plan and (b) section (ASCE, 1975).

and, in metric units:

$$d_{50} = \frac{0.027\ (Q)^{4/3}}{TW\ (D_0)} \qquad (9\text{-}8b)$$

where Y = depth of scour hole below culvert invert

(c) The other riprap or gabion requirements are as indicated in the previous sections for channel lining.
(d) Aprons constructed of man-made materials are often a viable alternative. Refer to the above discussion of man-made materials for design consideration.

IV. CHECK DAMS

A. Categories, Definitions, and Applications

Check dams are commonly used for energy dissipation and/or sediment control in streams. They are commonly constructed using gravel

or rock, gravel overlaying a sand core, timber, or gabions. One of the most important design considerations for check dams is the hazard to downstream properties posed by dam failure, and the designer cannot dismiss the problem as insignificant simply because "the dam is low" or "the volume of water stored is small."

Check dams constructed of earth fill are shown in Figure 9.15. These check dams are constructed of rock and soil materials which could be clean excavation spoil. The core of some of these structures contains finer size particles enclosed by coarse gravel, cobbles, and small boulders. Depending on the design, the check dam may be porous and act entirely as a filter, contain an outlet structure, or operate as a filter and overflow structure in combination. Both weirs and drop inlets may be used. Where large storage is anticipated in the impoundment created

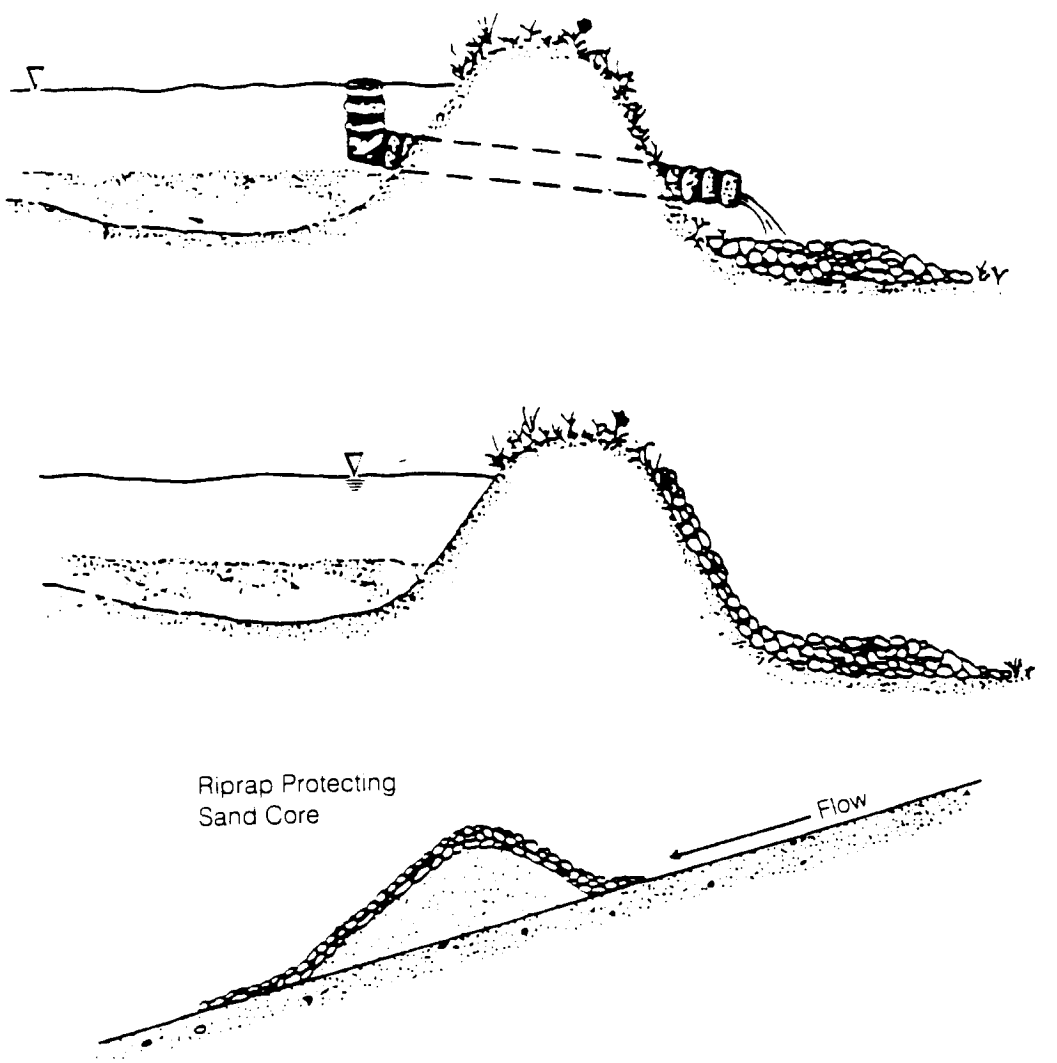


Figure 9.15—Alternative earthfill check dam designs (Shown for concept only.) (Barfield et al., *1983).*

by earth fill check dams, design procedures delineated in U.S. Bureau of Reclamation (1977) and Chapter 11 should be followed.

Check dams can change the flow regime from supercritical to subcritical when placed at appropriate intervals along the channel. The intervals are determined by dividing the steep channel into reaches in which backwater conditions are created by the dams.

B. Porous Check Dams

Porous check dams are primarily used for sediment trapping, however they can also be used for energy dissipation in steeply sloped channels. Check dams designed for sediment control can be used temporarily during construction, or as a permanent measure in watersheds that are subject to significant erosion and channel degradation. Check dams can be used as filters and impoundment structures that trap the sediments, or as a means of promoting aggradation in the channel by dissipating the flow energy and settling sediments.

Rock check dams may have large void spaces, therefore, the use of rocks in combination with smaller stones such as gravel or with filter cloth anchored to the check dam surface increases sediment trap efficiency. Gabions can also be used in check dams as shown in Figure 9.16. Gabions may be used strictly as porous dams, or as dam/weir combinations.

C. Impervious Check Dams

Impervious check dams are primarily used for energy dissipation and water level control upstream of the dam. Erosion and scour can be controlled in channels with steep slopes, and the degradation process replaced by aggradation. As shown in Figure 9.17, a series of check dams can be used to change the flow regime from supercritical to subcritical while providing backwater ponding and quiescent settling conditions.

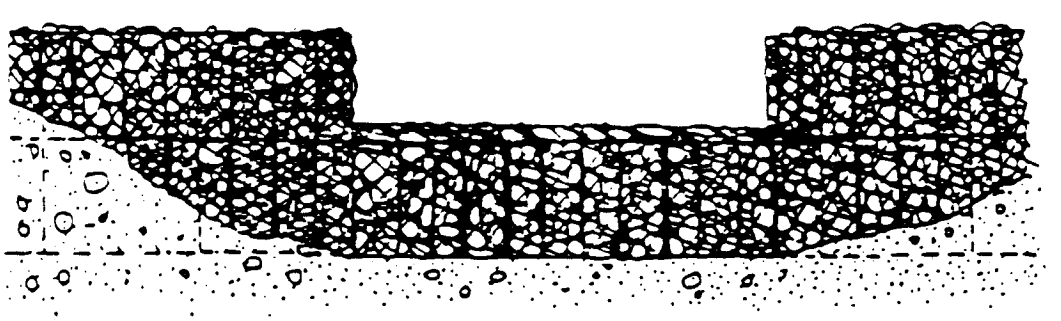


Figure 9.16—Gabion check dam (Barfield et al., *1983).*

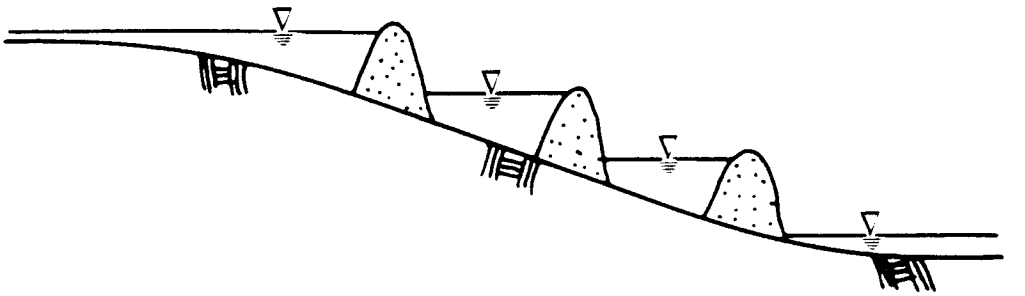


Figure 9.17—A series of check dams. (Shown for concept only.)

D. Check Dam Design Considerations

Porous check dams are designed to maximize sediment trap efficiency. The efficiency depends on the relationship between the size distribution of the sediment inflow and the voids in the dam. A sediment check dam is usually designed to provide quiescent flow and settling conditions by creating backwater detention ponds. A porous check dam design methodology developed by Hirschi (undated) is based on backwater surface profiles and Stokes' Law. This methodology considers the following factors:

(a) Inflow sediment size distribution.
(b) Design flow.
(c) Check dam porosity.
(d) Channel slope.
(e) Manning's "n."
(f) Channel configurations.

The design analysis may include the following steps:

(a) Determination of the backwater curve.
(b) Designation of the quiescent settling zone.
(c) Calculation of the lateral distance available for settling.
(d) Estimation of flow velocity in the stream.
(e) Estimation of travel time.
(f) Estimation of flow depths.
(g) Estimation of trap efficiency.

Impervious check dam designs generally are based on the same considerations, except for the steps related to sediment trap efficiency, and require the following additional design elements.

(a) Minimum tailwater at the dam.
(b) Overflow weir level that provides minimum tailwater and results in the desired backwater surface profile.
(c) Flow velocity that is below maximum permissible velocity in the channel.

Maintenance at check dams normally includes dredging and off-site disposal. The design should estimate the frequency of dredging required before assuming that check dams are feasible.

V. ENERGY DISSIPATORS

A. Definitions, Applications, and Categories

Energy dissipators are required in the immediate vicinity of hydraulic structures where high impact loads, erosive forces and severe scour are expected. Said another way, they are usually required where the flow regime changes from supercritical to subcritical or where the flow is supercritical and the tractive force or flow velocities are higher than the maximum allowable values. The basic hydraulic parameter that identifies the flow regime, and that is used in connection with energy dissipators in general, and with hydraulic jump dissipators in particular, is the Froude number. A Froude number equal to one indicates a critical flow condition, while numbers greater or less than one indicate supercritical or subcritical flow, respectively. The Froude number (F) is estimated using the following equation (Rouse 1949):

$$F = \frac{v}{\sqrt{gy}} \tag{9-9}$$

Where:

v = flow velocity
g = acceleration due to gravity
y = hydraulic mean depth (cross-section area/top width)

The Froude number is a product of the flow velocity and the wave celerity. In rectangular channels and some large rivers, the equation may be rewritten in the following form:

$$F^2 = \frac{1}{B}\left(\frac{Q^2}{gy^3}\right) \tag{9-10}$$

Where:

Q = discharge
B = width of channel

Energy dissipation structures act as transitions which reduce high flow velocities that may exist under a range of flows. The energy dis-

Denver, Colorado—Innovative energy dissipator in an open channel.

sipators localize hydraulic jumps and act as stilling basins. The use of energy dissipators is very common downstream of hydraulic structures where common channel protection cannot be used alone because of potential damage. If riprap or other protection is used for energy dissipation, it should be confined in a basin and secured in place with grout or mesh.

Energy dissipators range from simple horizontal concrete aprons and hydraulic jump basins to wave suppressors and flip buckets for large streams and structures as described in Peterka (1958) and McLaughlin Water Engineers (1986). Because of their special hydraulic characteristics, drop structures are discussed later in this chapter. The primary difference between energy dissipators in general and drop structures is that the energy dissipators are used to reduce high velocities at critical locations by hydraulic jumps, while drop structures are vertical structures used for controlling velocities in channel reaches by reducing channel slopes. Because of various appurtenances such as sills, baffles, weirs, and because of variations in the geometry of the stilling basins, a wide variety of energy dissipators is used, and a number of them are discussed below.

B. Riprap Basins for Small Culvert Outlets

The most common energy dissipators used for stormwater management are riprap basins (Figure 9.18). Their advantages include simplicity, low cost, and wide application.

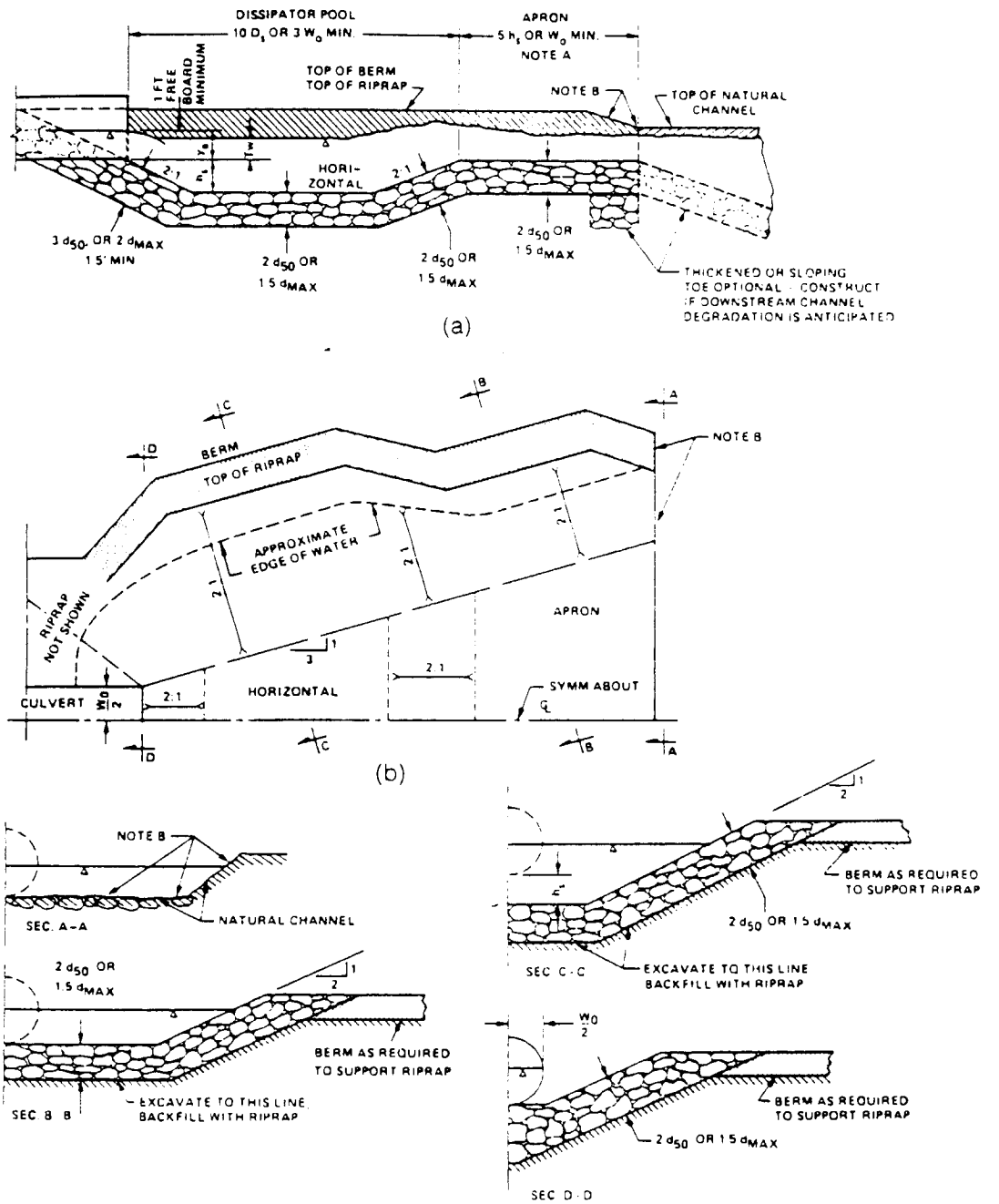


Figure 9.18—Typical riprap basin: (a) centerline section and (b) half plan: W_o = diameter for pipe culvert, W_o = barrel width for box culvert, and W_o = span of pipe-arch culvert. Note A: If exit velocity of basin is specified, extend basin as required to obtain sufficient cross-sectional area at section A-A such that Q_{des}/(cross section area at sec. A-A) = specified exit velocity. Note B: Warp basin to conform to natural stream channel. Top of riprap in floor of basin should be at the same elevation or lower than natural channel bottom at sec. A-A (U.S. Federal Highway Administration, 1983).

The riprap placed in the basin must be inspected and repaired, if necessary, after major storms. The median stone diameter can be estimated based on the exit velocity of the pipe or culvert as shown in McLaughlin Water Engineers (1986) and the AASHTO Drainage Handbook (1987). The length of the basin is estimated based on the width or diameter of the conduit. The depth of the basin is based on the median stone diameter.

C. Stilling Basins

If a hydraulic jump is used for energy dissipation, it should be confined to a heavily-armored channel reach, the bottom of which is protected by a solid surface such as concrete to resist scouring. Since the cost of concrete structures is relatively high, the length of the hydraulic jump is usually controlled by accessories that not only stabilize the jump action and increase the factor of safety, but reduce the cost of the structure.

1. Design Considerations

There are several considerations that should be included in designing hydraulic jumps and stilling basins (Chow 1959; U.S. D.O.T., 1983):

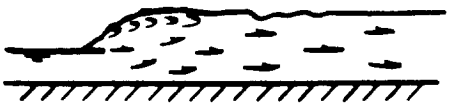
(a) Jump Position—There are three positions or alternative patterns that allow a hydraulic jump to form downstream of the transition in the channel. These positions are controlled by tailwater.
(b) Tailwater Conditions—Tailwater fluctuations due to changes in discharge complicate the design procedure. They should be taken into account by classification of tailwater conditions using tailwater and hydraulic jump rating curves (see also Chapter 6).
(c) Jump Types—U.S. Bureau of Reclamation (1955) and Bradley and Peterka (1957) identify various types of hydraulic jumps that may occur. These are summarized in Figure 9.19. Oscillating jumps in a Froude number range of 2.5 to 4.5 are best avoided, or specially designed wave suppressors may be used to reduce wave impact.

The greater the Froude number the higher the effect of tailwater on the jump. Therefore for a Froude number as low as 8 the tailwater depth should be greater than the sequent depth downstream of the jump so that the jump will stay on the apron. When the Froude number is greater than 10, the common stilling basin dissipator may not be as cost-effective as a special bucket type dissipator (see Peterka 1958).

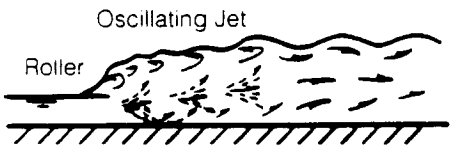
2. Control of Jumps

Jumps can be controlled by several types of appurtenances such as sills, chute blocks, and baffle piers. The purpose of a sill located at the end of a stilling basin is to induce jump formation and to control its position under most probable operating conditions. Sharp crested or broad crested weirs can be used to stabilize and control the jump.

F_1 = 1–1.7 Undular Jump



F_1 = 1.7–2.5 Weak Jump



F_1 = 2.5–4.5 Oscillating Jump

F_1 = 4.5–9.0 Steady Jump

F_1 >9.0 Strong Jump

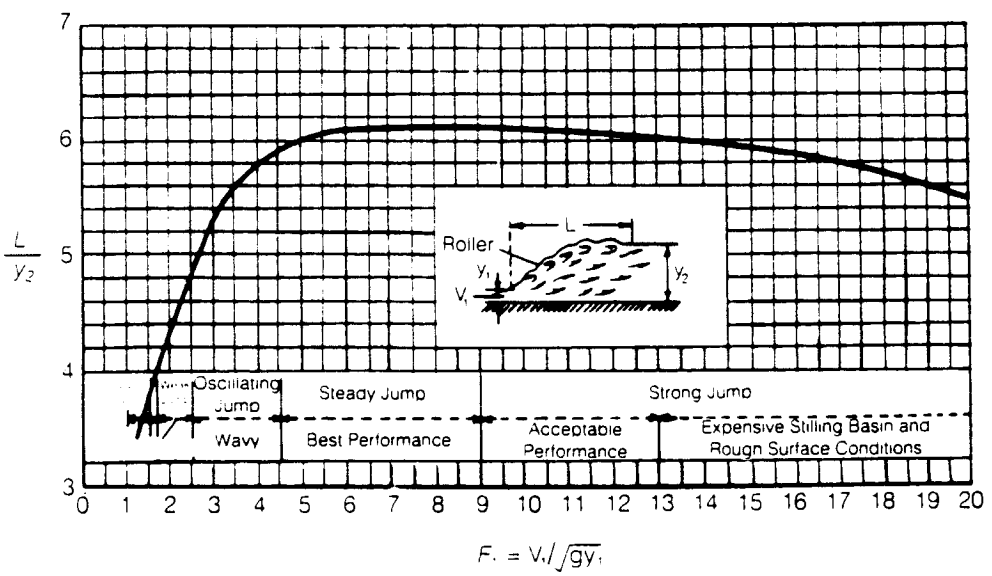


Figure 9.19—Lengths and types of hydraulic jumps in horizontal channels (Bradley and Peterka, 1957; Chow, 1959).

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Chute blocks are used at the entrance to the stilling basin. Their function is to furrow the incoming jet and lift a portion of it from the floor, producing a shorter length of jump than would occur without them.

Baffle piers are blocks placed in intermediate positions across the basin floor for dissipating energy mostly by direct impact action. They are useful for small structures with low flow velocities. High flow velocities may result in cavitation action on the piers and basin floor downstream.

3. Stilling Basin Categories

The following three major categories of basins are used for a range of hydraulic conditions. Design details can be found in the AASHTO Drainage Handbook (1987), Chow (1959), and U.S. D.O.T. (1983).

(a) The SAF ("Saint Anthony Falls") Stilling Basin (Chow 1959)—This basin, shown in Figure 9.20, is recommended for use on small structures such as spillways and outlet works where the Froude number varies between 1.7 and 17. The appurtenances used for this dissipator can reduce the length of the basin by approximately 80%. This design has great potential in stormwater management because of its applicability to small structures.

Stilling Basin III developed by the U.S. Bureau of Reclamation (USBR) is similar to the SAF basin, but it has a higher factor of safety.

(b) The USBR Stilling Basin II—This basin, shown in Figure 9.21, is recommended for use with jumps with Froude numbers greater than 4.5 at large spillways and canals. This basin may reduce the length of the jump by a third. This basin is used for high-dam and earth-dam spillways. Appurtenances used in this basin include chute blocks at the upstream end of the basin and a dentated sill at the downstream end. No baffle piers are used in this basin because of the cavitation potential.

(c) The USBR Stilling Basin IV—This basin (Figure 9.22) is used where jumps are imperfect or where oscillating waves occur with Froude numbers between 2.5–4.5. This design reduces excessive waves by eliminating the wave at its source through deflection of directional jets using chute blocks. When a horizontal stilling basin is constructed without appurtenances, the length of the basin is made equal to the length of the jump.

D. Simple Energy-Dissipating Headwalls

Another simple type of energy dissipator that can be used at culvert outlets are energy dissipating headwalls. Three typical such headwalls are shown in Figures 9.23 through 9.25.

E. Design Criteria and Practices

Most of the design criteria for stilling basin dissipators are included in the previous paragraphs. Table 9.8 provides a summary of selected

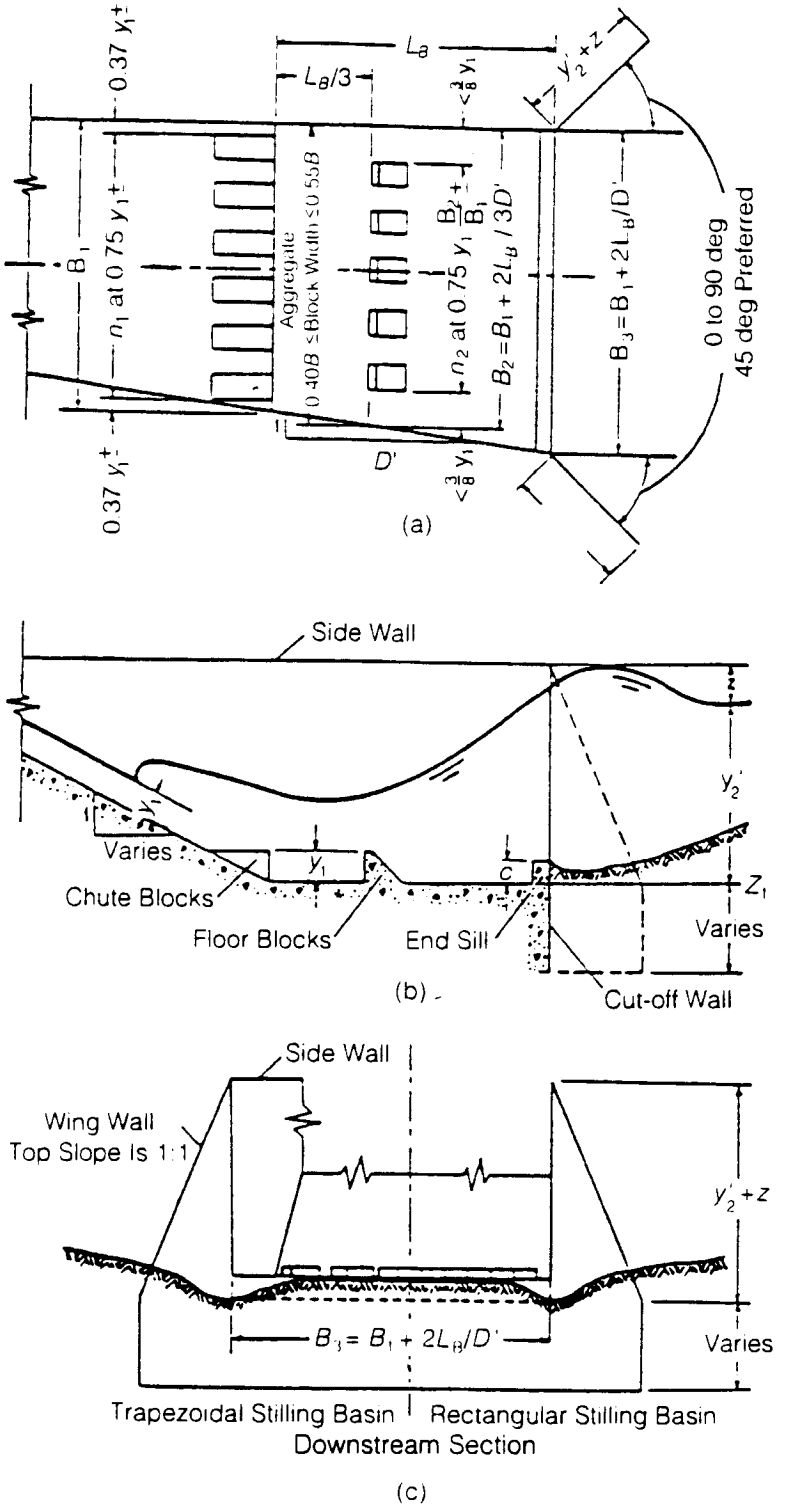


Figure 9.20—Proportions of the SAF basin: (a) rectangular stilling basin half-plan, (b) trapezoidal stilling basin elevation, and (c) centerline section (Chow, 1959).

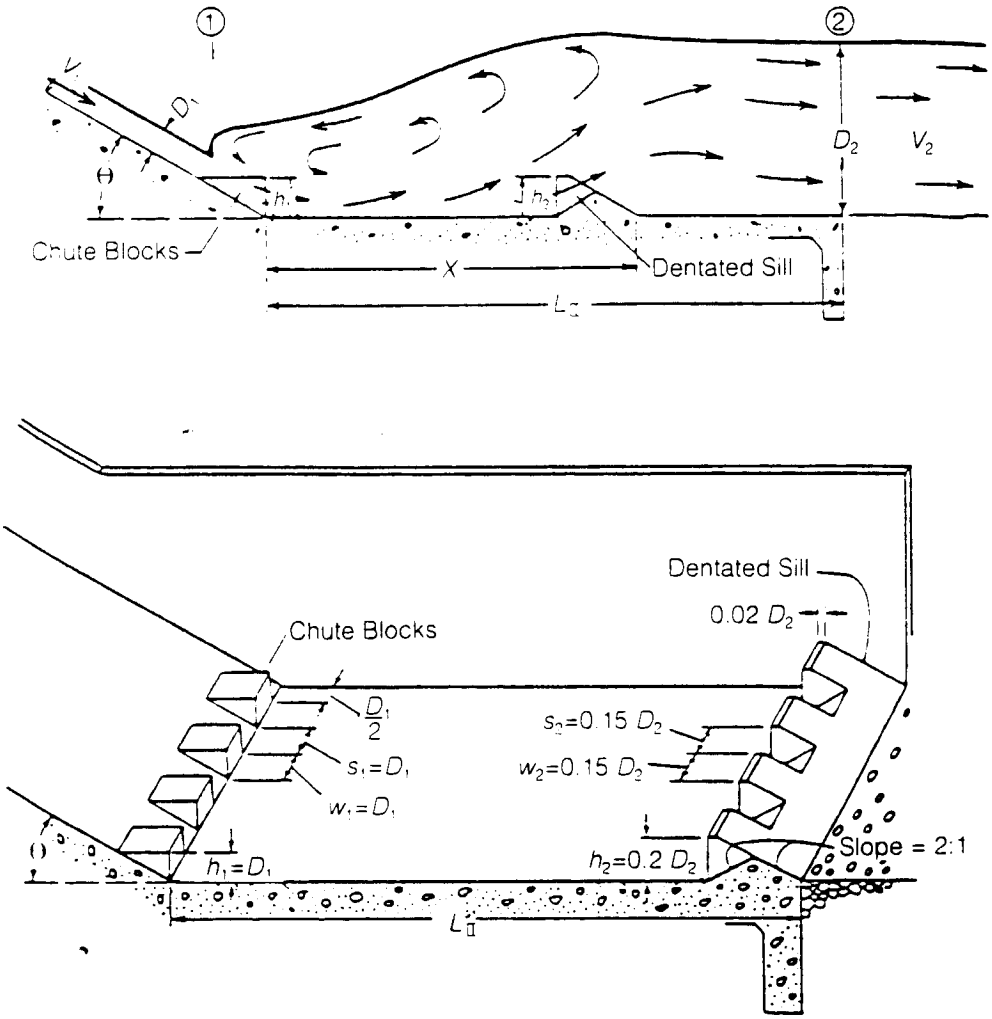


Figure 9.21—Proportions of the USBR basin II (Chow, 1959).

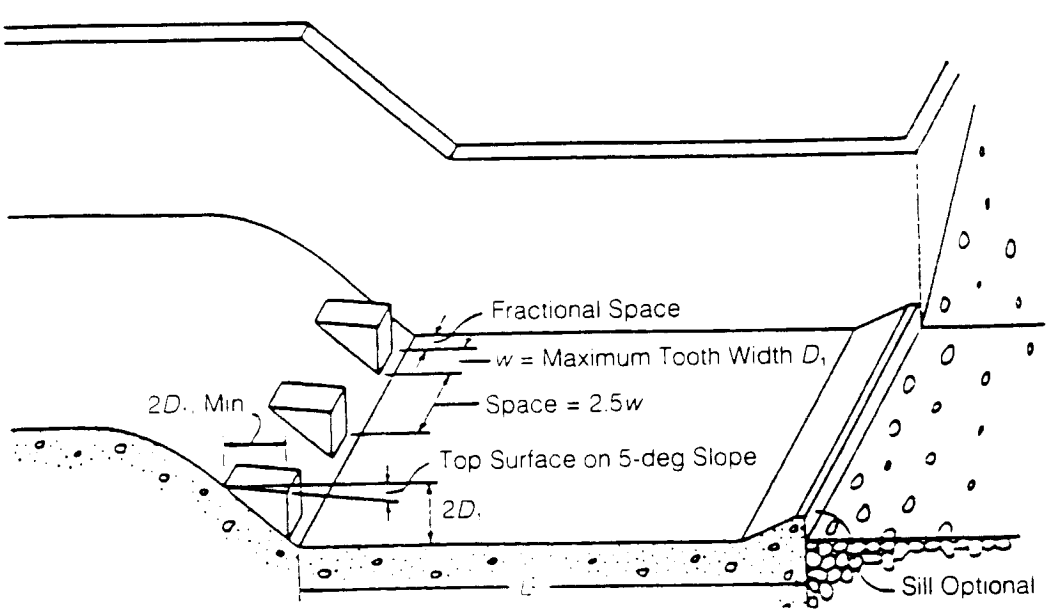


Figure 9.22—Proportions of the USBR basin IV (Chow, 1959).

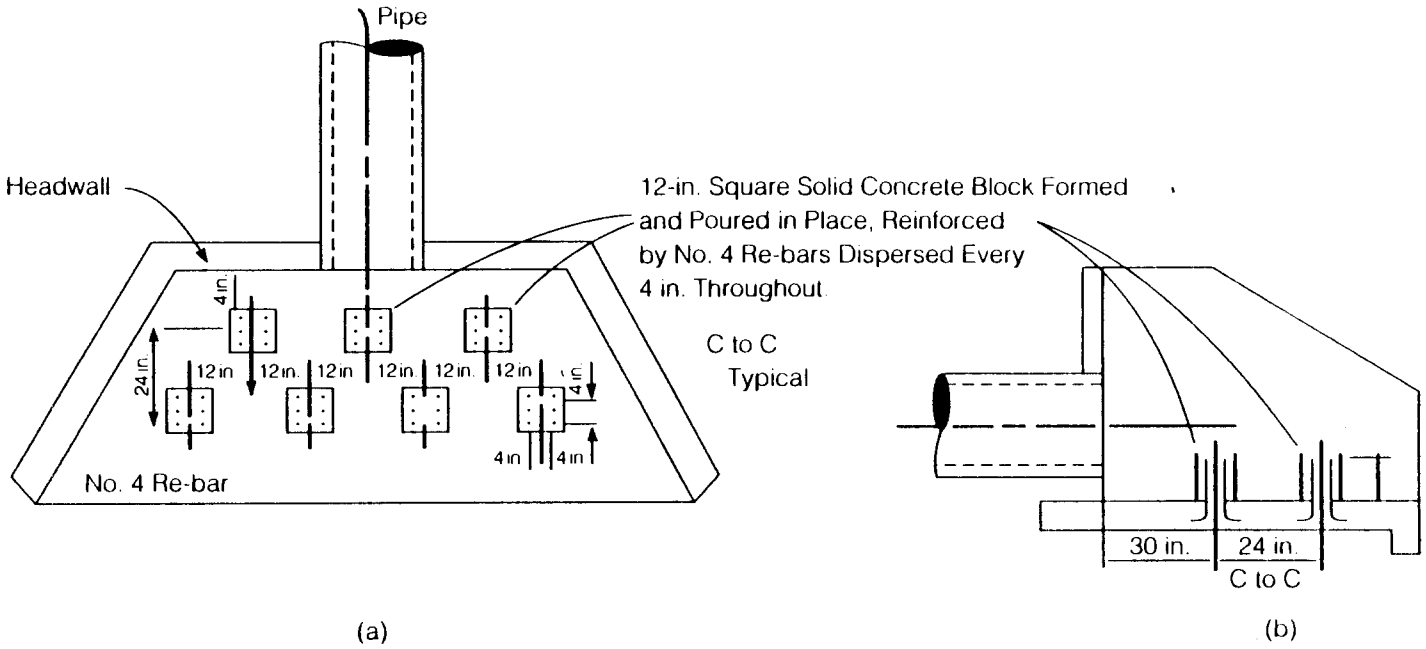


Figure 9.23—Standard energy dissipating headwall, type I: (a) plan, and (b) side elevation (in. × 25.4 = mm).

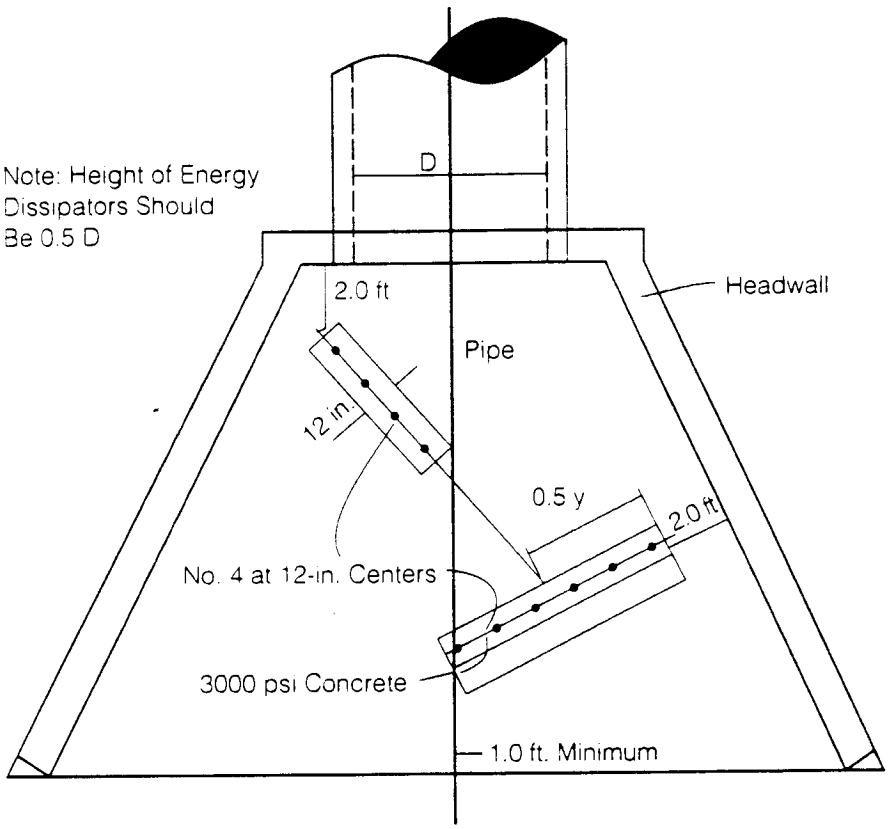
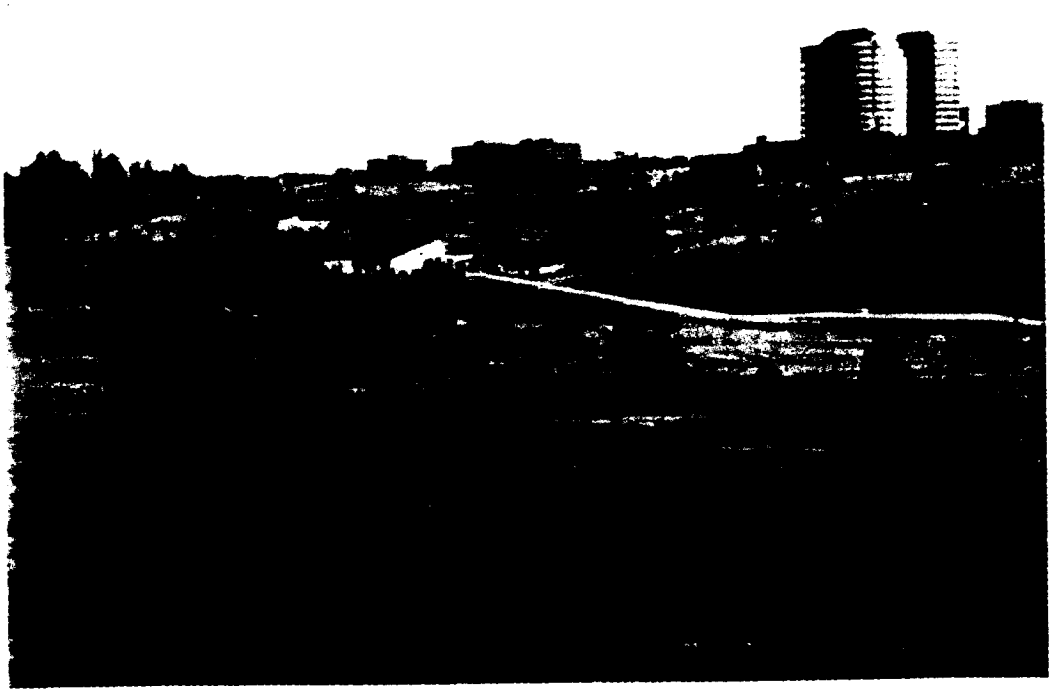


Figure 9.24—Standard energy dissipating headwall, Type II.



Drop/grade control structures sculptured to fit the urban park setting.

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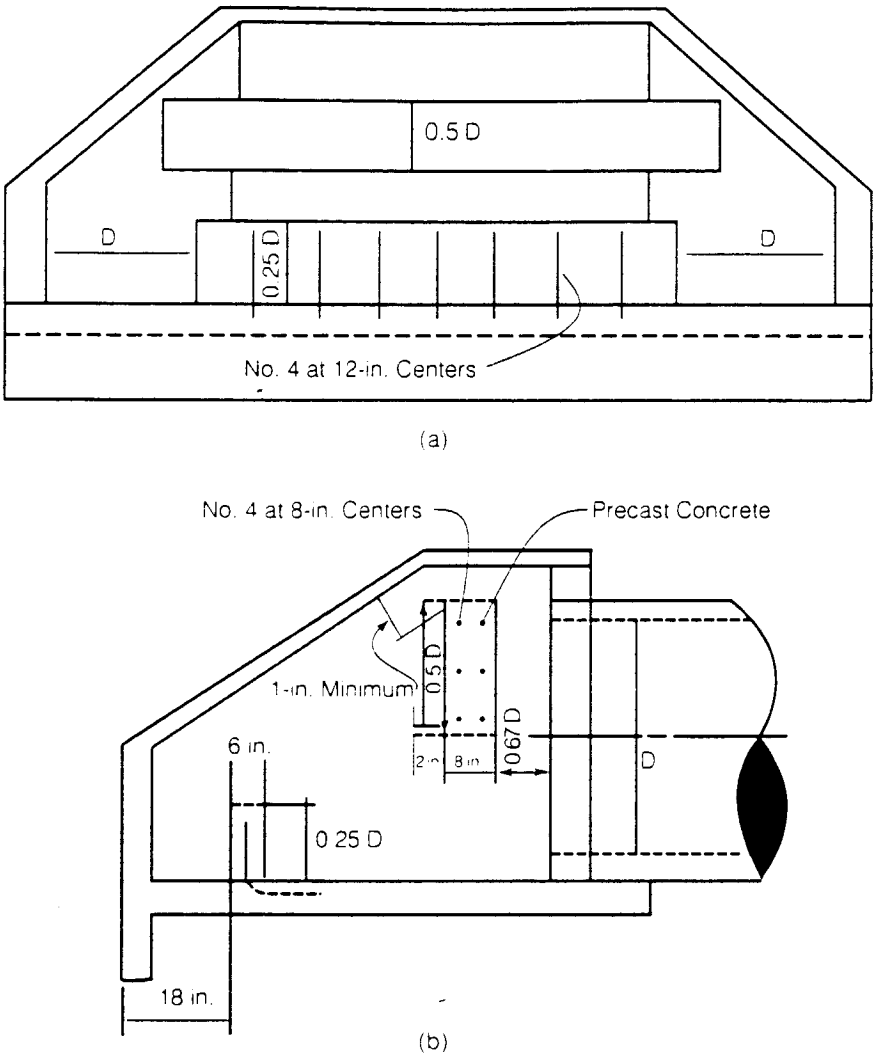


Figure 9.25—Standard energy dissipating headwall, Type III: (a) front elevation and (b) side elevation.

parameters, and may be used for preliminary identification of alternative energy dissipators that may used. Because of the great variety and combination of types of energy dissipators and appurtenances, the designer should review available references in sufficient detail to arrive at a design that is suited to his specific field conditions.

VI. DROP STRUCTURES

A. Definitions, Categories, and Applications

Vertical drop structures are controlled transitions for energy dissipation in steep channels where riprap or other energy dissipation structures are not as cost effective. Drop structures used for drainage and

TABLE 9.8. Dissipator Criteria (U.S. Department of Transportation 1983)

Dissipator Type	Froude Number *Fr*	Allowable Debris			Tailwater *TW*	Special Consideration
		Silt Sand	Boulders	Floating		
Free Hydraulic Jump	>1	H	H	H	Required	
CSU Rigid Boundary	<3	M	L	M	—	
Tumbling Flow	>1	M	L	L	—	$4 < S_o < 25$
Increased Resistance	—	M	L	L	—	Check Outlet Control *HW*
USBR Type II	4 to 14	M	L	M	Required	
USBR Type III	4.5 to 17	M	L	M	Required	
USBR Type IV	2.5 to 4.5	M	L	M	Required	
SAF	1.7 to 17	M	L	M	Required	
Contra Costa	<3	H	M	M	<0.5*D*	
Hook	1.8 to 3	H	M	M	—	
USBR Type VI	—	M	L	L	Desirable	$Q < 400$ cfs, $V < 50$ fps
Forest Service	—	M	L	L	Desirable	$D < 36$ inch
Drop Structure	<1	H	L	M	Required	Drop < 15 ft.
Manifold	—	M	N	N	Desirable	Note:
Corps Stilling Well	—	M	L	N	Desirable	N = none L = low
Riprap	<3	H	H	H	—	M = moderate H = heavy

storm water management can be categorized primarily as either open channel transitions (drop spillways) or transitions between open channel and closed conduits (drop shafts).

Drop structures are usually constructed of concrete because of the forces involved, however, it is possible to utilize riprap stilling basins or gabions where physical, economic and other conditions permit.

B. Open Channel Drops (Drop Spillways)

Drop structures in open channels change the channel slope from steep to mild by combining a series of gentle slopes and vertical drops. Flow velocities are reduced to non-erosive velocities, while the kinetic energy or flow velocity gained by the water as it drops over the crest of each spillway is dissipated by an apron or stilling basin.

Open channel drop structures generally require aerated nappes and subcritical flow conditions at both the upstream and downstream section of the drop. The stilling basin can vary from a simple concrete apron to baffle blocks or sills as described previously.

Figure 9.26 shows the flow geometry and important variables at a vertical (straight) drop structure. The flow geometry at such drops can be described by the drop number (D) which is defined (Chow 1959) as:

$$D = \frac{q^2}{gh^3} \tag{9-11}$$

Where:

q = discharge per unit width of crest overfall, cfs/ft
g = acceleration of gravity, ft/sec^2
h = height of the drop, ft

The drop functions are:

$$\frac{L_d}{h} = 4.30D^{0.27} \tag{9-12}$$

$$\frac{Y_p}{h} = 1.00D^{0.22} \tag{9-13}$$

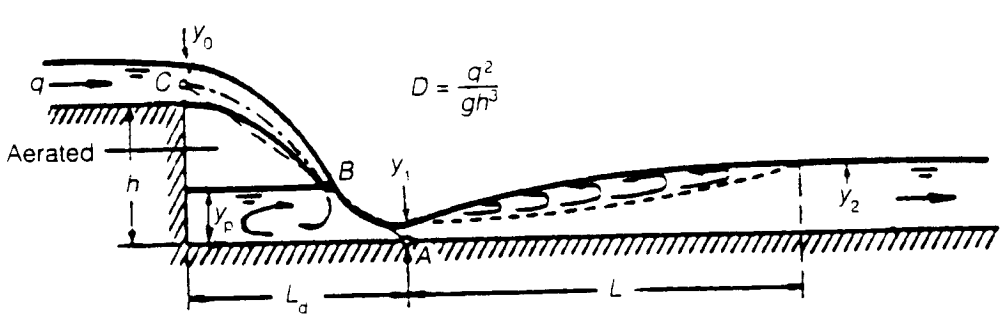


Figure 9.26—Flow geometry of a straight drop spillway (Chow, 1959).

$$\frac{Y_1}{h} = 0.54D^{0.425} \tag{9-14}$$

$$\frac{Y_2}{h} = 1.66D^{0.27} \tag{9-15}$$

Where:

L_d = drop length
Y_p = pool depth under the nappe
Y_1 = the depth of the toe of nappe
Y_2 = tailwater depth sequent to Y_1

For a given drop height, h, and discharge, q, the drop length L_d and the sequent depth Y_2 can be estimated by Equations 9-12 and 9-15, respectively. The length of the jump can be estimated by techniques discussed in Section V. If the tailwater is less than Y_2, the hydraulic jump will recede downstream. Conversely, if the tailwater is greater than Y_2, the jump will be submerged. If tailwater is equal to Y_2 no supercritical flow exists on the apron and the distance L_d is minimum.

When the tailwater depth is less than Y_2 it is necessary (according to U.S. Department of Transportation 1983) to provide either 1) an apron at the bed level and a sill or baffles, or 2) an apron below the downstream bed level and an end sill.

The choice of design type and dimensions depends on the unit discharge (q), drop height (h), and tailwater depth. The design should take into consideration the geometry of the undisturbed flow. If the spillway (overflow crest) length is less than the width of the approach channel, the approach channel must be designed properly to reduce the effect of the end contractions to avoid scour.

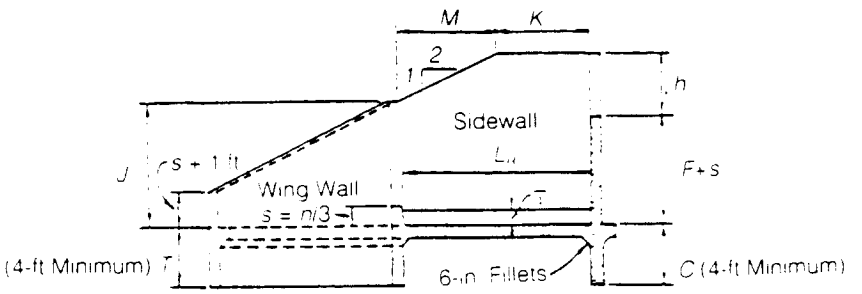
The two most common vertical open channel drops are the straight drop structure and the box inlet drop structure.

1. *Straight Drop Structure*

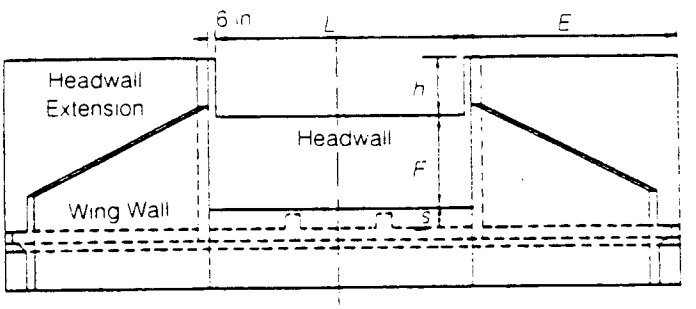
Figure 9.27 shows a layout of typical straight drop structure and hydraulic design criteria developed by the U.S. Soil Conservation Service. McLaughlin Water Engineers (1983) provides specific criteria and reviews design considerations related to hydraulic, geotechnical, and structural design of drop structures.

2. *Box Inlet Drop Structure*

The box inlet drop structure is a rectangular box open at the top and downstream end as shown in Figure 9.28 (U.S. D.O.T. 1983). Water is directed to the crest of the box inlet by earth dikes and a headwall. Flow enters over the upstream end and two sides. The long crest of the box inlet permits large flows to pass at relatively low heads. The



(a)



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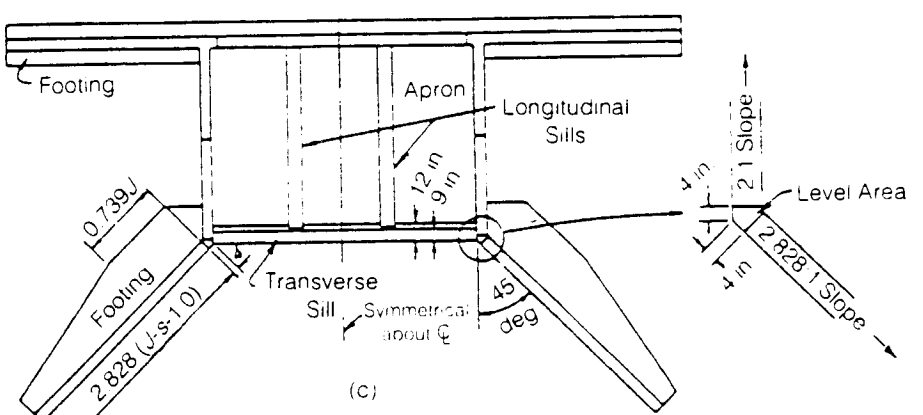

(c)

E = Minimum length of headwall extension (ft) = $[3h+2]$ or $[1.5F]$, whichever is greater

J = Height of wing wall and sidewall at junction (ft) = $[2h]$ or $\left[F+h+s-\left(\frac{L_B+0.42}{2}\right)\right]$ or $[t+1]$, whichever is greater

L_B = Length of basin (ft) = $[F(2.28\frac{h}{F}+0.52)]$

M = $[2(F+1.3h-J)]$

K = $[(L_B+0.42)-M]$

(d)

Figure 9.27—Typical drop spillway and some hydraulic design criteria: (a) section on center line, (b) downstream elevation, (c) plan, and (d) criteria (in. × 25.4 = mm and ft × 0.304 8 = m) (U.S. Soil Conservation Service, 1954).

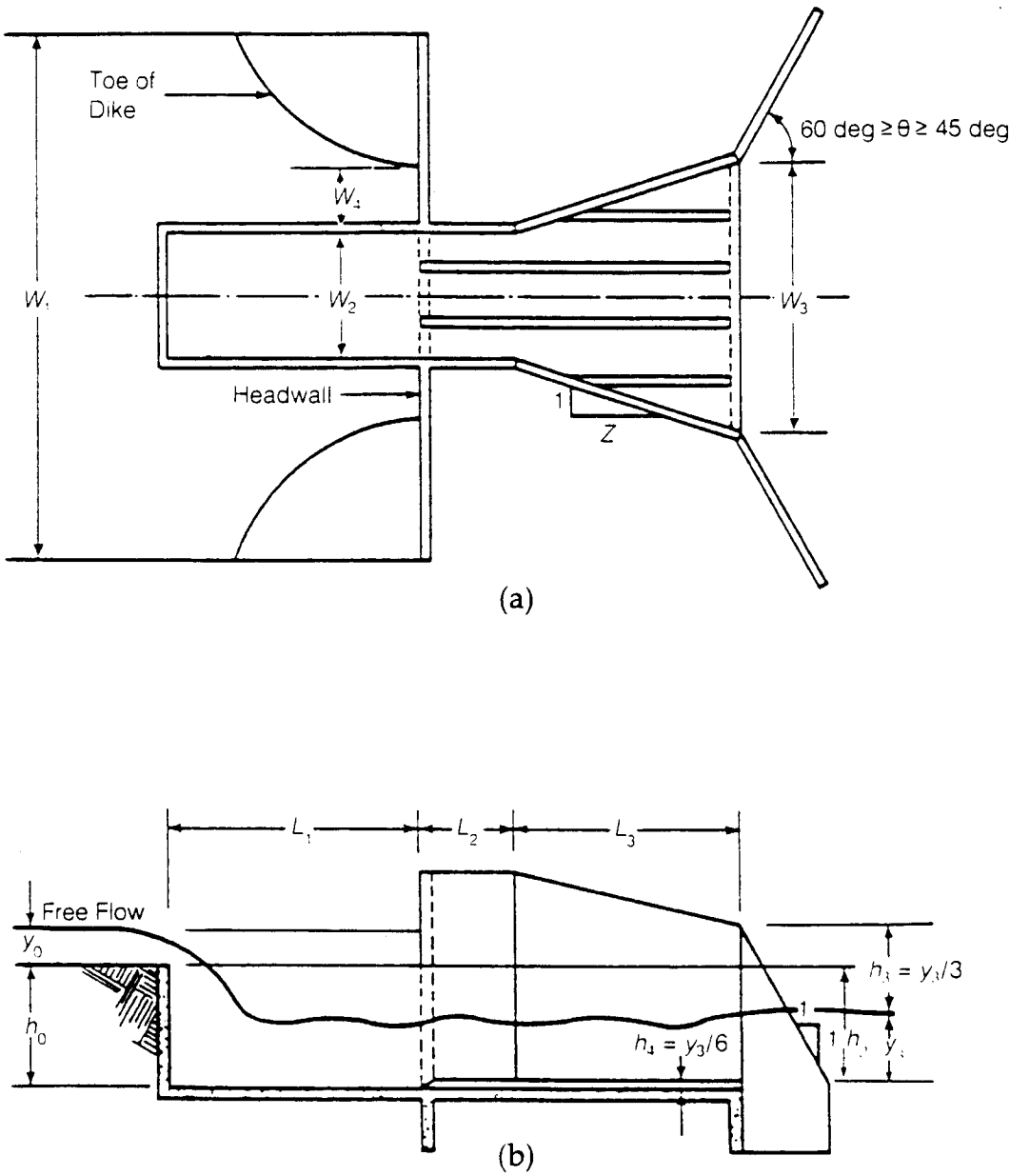

Figure 9.28—Box inlet drop structure: (a) plan, and (b) section on center line (U.S. Dept. of Transportation, 1983).

width of the structure should not be greater than the downstream channel. Box inlet drop structures are applicable to drops from 2–12 feet.

Design data and criteria for these structures based on U.S. Soil Conservation Service and St. Anthony Falls Hydraulic Laboratory are available (U.S. Department of Transportation 1983; Blaisdell and Donnelly 1956). The parameters to consider for the hydraulic design of the drop are:

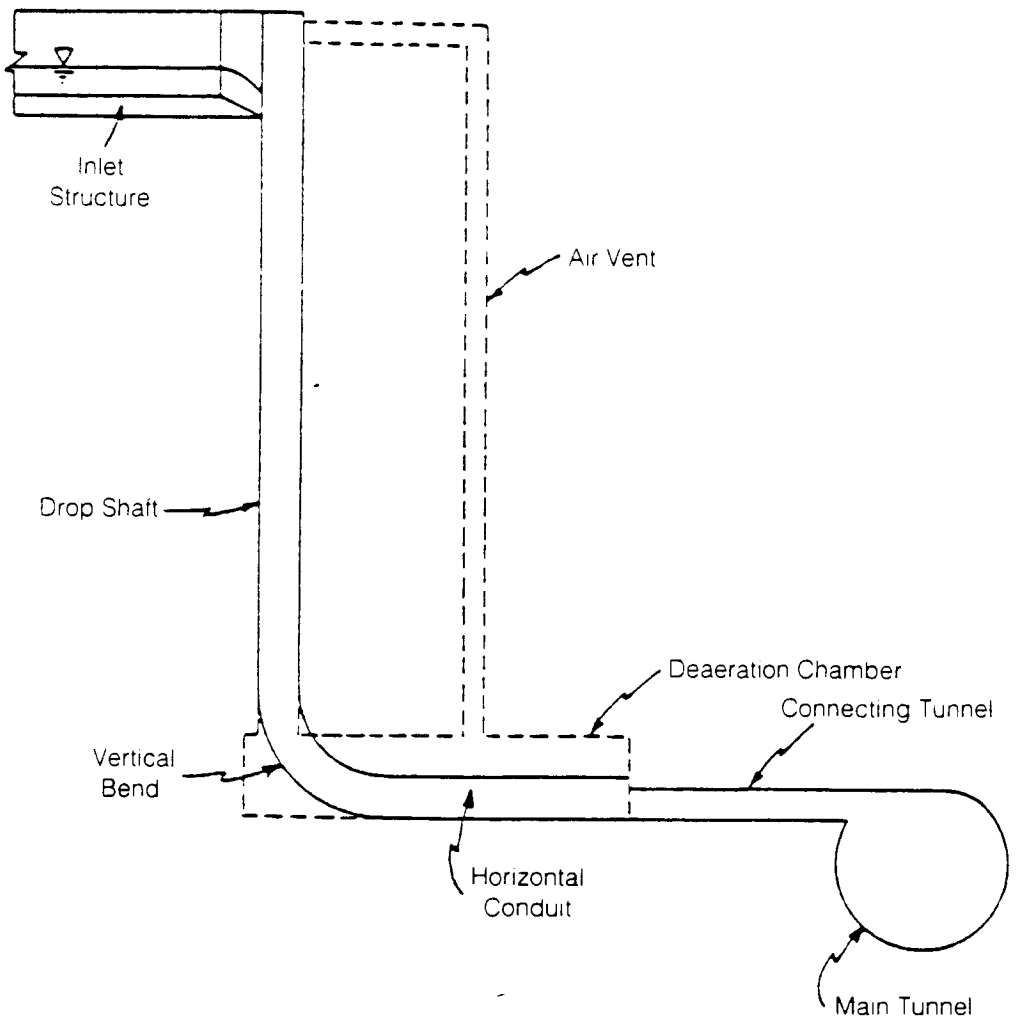

Figure 9.29—Components of a typical drop structure (Jain and Kennedy, 1983).

(a) Section (length) of the crease of the box inlet
(b) Opening the headwalls
(c) Discharge, discharge coefficients and flow regime changes
(d) Box inlet length and depth
(e) Minimum length and width of stilling basin

C. Drop Shaft Structures

Storm sewer drop shafts are commonly used to dispose of storm runoff in deep large pipes or tunnels. They consist of an inlet structure, the drop shaft, vertical bend, horizontal conduit with or without a deaeration chamber, outlet structure and auxiliary structures. Schematics of most of these components are shown in Figure 9.29 (Jain and Kennedy 1983). The drop shaft structure design can vary tremendously.

According to Bushey (1952), there are three methods of dropping flows down a deep vertical shaft (Figure 9.30).

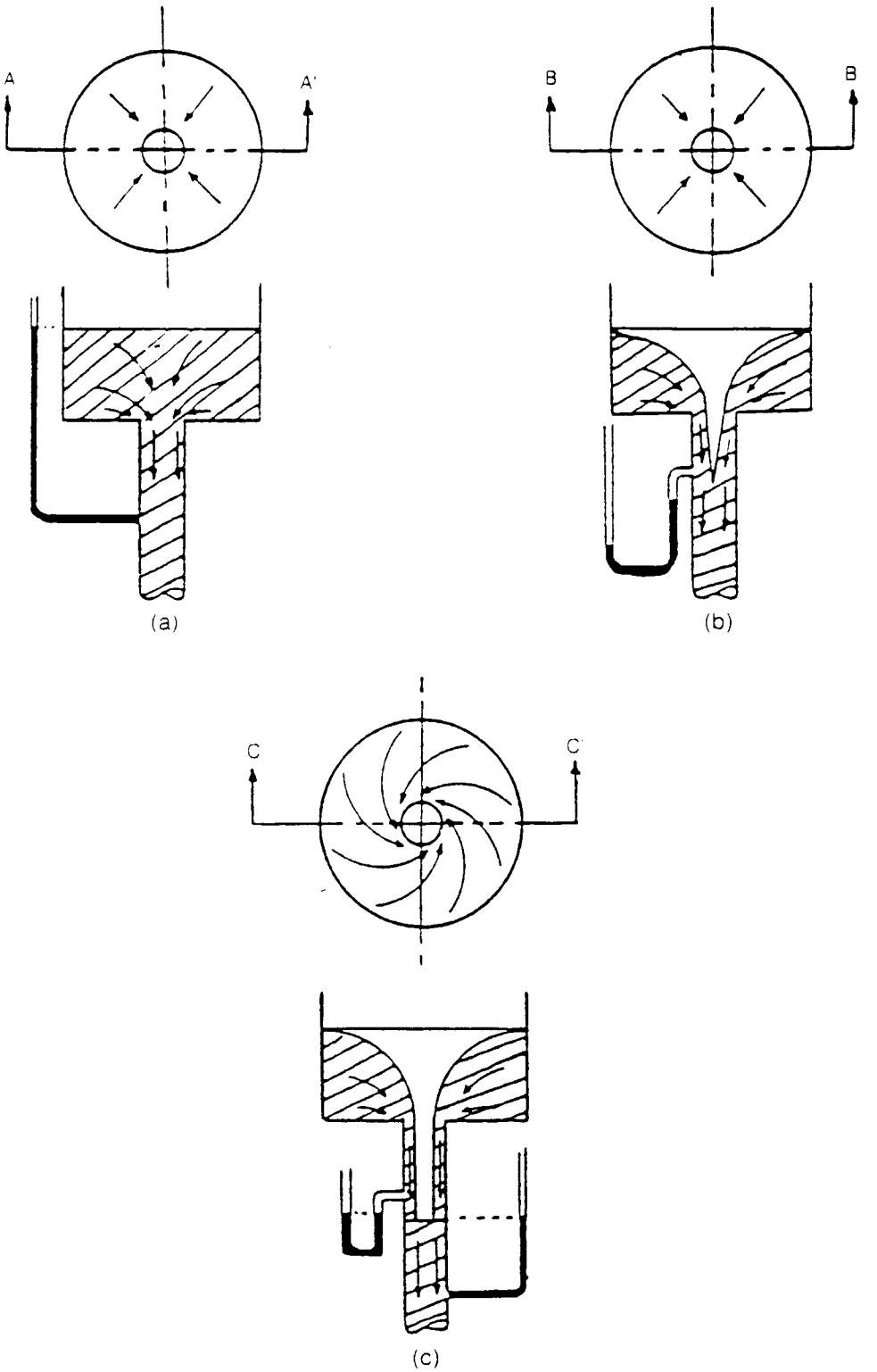

Figure 9.30—Types of flow drops in shafts. (a) Completely flooded shafts, (b) subatmospheric drop shafts, and (c) vortex drop shafts (Bushey, 1952).

1. *Completely Flooded Drop Shafts*

This type of structure has an inlet tank and a vertical shaft and has the hydraulic grade line above the top of the shaft. Under this condition, pressures throughout the structure are hydrostatic and the water does not fall freely. Changes in pressure in the conduit or tunnel connected to the drop are transmitted through the column of water in the shaft, and result in an immediate corresponding change in the water depth at the inlet tank. With this design the water surface in the tank remains level, there is no drawdown in the shaft, and air cannot be entrained. The discharge is easily computed by Bernoullis' equation and conventional methods of pipe flow analysis.

2. *Subatmospheric Drop Shafts*

This type of drop shaft is characterized by a radial flow toward the shaft. The water surface is drawn down as the velocity of the flow increases in the approach toward the center of the tank. Large quantities of air are entrained as the water drops into the shaft. The water appears to be mixed with air and there are also voids in the freely-falling water. The pressure at the shaft inlet is subatmospheric. This negative pressure results in large discharge under low head in the tank. However, since these negative pressures are the source of air entrainment and cavitation, this method of entry is considered undesirable. The subatomspheric drop shaft results in unsteady flow with large slugs of air breathed in, which result in considerable noise. To achieve radial flow, it is important that the entrance to the tank have radial fins or guide vanes. A bell-mouth or a morning-glory entrance to the shaft can improve the hydraulic performance of the system.

The U.S. Department of Transportation (1983) has noted the following undesirable characteristics of radial entrances, and suggests that radial flow be avoided.

(a) Negative pressure near the inlet of the shaft.
(b) Intake of large quantities of air.
(c) Need for radial guide vanes to provide steady motion, depth and pressure.

If radial fins or guide vanes are not used, the radial flow may change to spiral or vortex flow.

3. *Spiral (Vortex) Drop Shafts*

This type of drop shaft is characterized by spiraling water that flows around the shaft walls with an air void that is connected to the atmosphere and extends into the shaft down to the hydraulic grade line. The pressure in this region is essentially atmospheric. Air is drawn into the shaft constantly, however, it is drawn in at a lower rate than with radial flow at the same discharge.

The undesirable conditions created by the radial entrance are avoided if the spiral or vortex approach is used. The following conditions usually prevail at a vortex drop shaft:

(a) Pressures are always atmospheric above full shaft.
(b) Smaller quantities of air are entrained.
(c) The vortex is very stable and does not vary with time.

Since air entrainment is responsible for cavitation and impact on the integrity of the structure, the spiral or vortex drop shaft is recommended for use. Blaisdell and Donnelly (1956) provide a review of a variety of vortex drop shafts, modeling techniques, and design considerations. Figure 9.31 shows a plan and profile of one of the many types of vortex inlets.

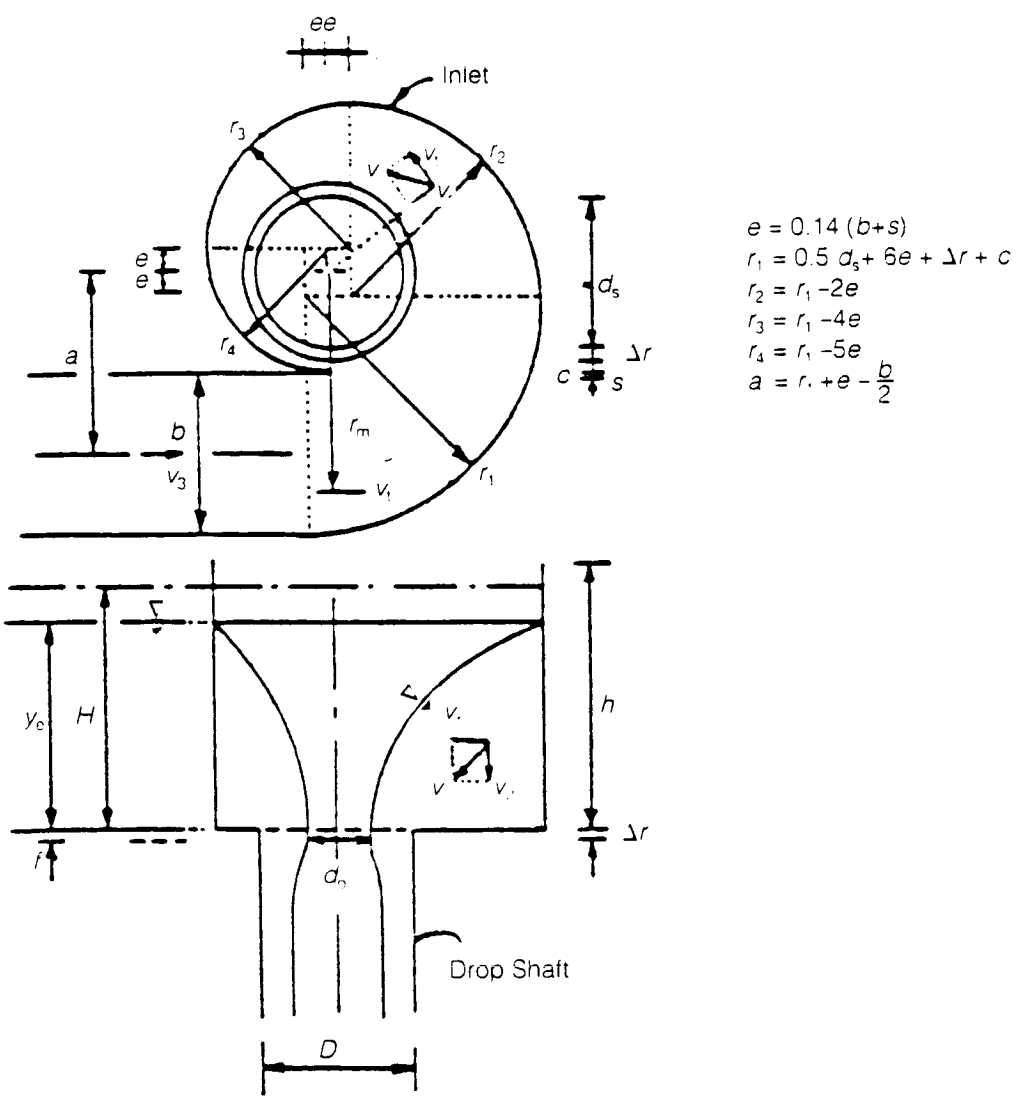

Figure 9.31—Standard scroll inlet for a vortex flow drop structure (Jain and Kennedy, 1983).

4. *Design Considerations*

The design of drop shafts focuses on the dimensions of the various components of the drop, and on maintaining the structural integrity of the system. Design charts are available from Bushey (1952), Jain and Kennedy (1983), and Anderson and Dahlin (1968). Structural integrity issues include the following:

(a) Elimination of potential cavitation that may result from air entrainment and subatmospheric pressure in the drop shafts. This can be done by using atmospheric pressure vortex drop shafts, keeping drop shaft heads low, and aerating the flow.
(b) Energy dissipation, which can be provided by:
  (1) Wall friction.
  (2) Annular hydraulic jumps produced by creating constrictions in the drop shaft.
  (3) Drop shaft baffles.
  (4) Plunge pools at the bottom of the shaft.
  (5) Impact cups located at the bottom of the shaft.
  (6) Wear-resistant linings such as iron-fiber reinforced concrete or stones.

Even though extensive literature is available on the subject, the designer should apply judgment relative to the applicability of available experimental data to specific design. Because of site conditions and design considerations not covered by previous experiments, physical hydraulic modeling may be required.

VII. SIPHONS

A. Definitions, Categories, and Applications

Any conduit that drops under an obstruction such as railroad tracks, depressed roadways or utilities, and regains elevation at the downstream side of the obstruction is referred to as a depressed sewer, or inverted siphon.

Because of the inverted bottom, the siphon stands full of storm water even when there is no flow. Some drainage districts discourage the use of siphons on the basis that the siphon requires more frequent maintenance including removal of debris that may clog the conduit. Nevertheless, siphons have advantages in particular settings, usually in urban areas where other solutions such as flow re-routing may result in disruptions to traffic and higher costs.

Siphons are normally single- or multi-barrel and consist of an entrance section, drop, depressed reach, rise, and outlet structure. Siphons require hydraulic head to operate properly and the adequacy of available head should be assessed early in the design process. Siphons can be simple or sophisticated, and the related design effort can be nominal or complex. The following examples apply to larger, sophisticated siphons with multiple appurtenances—some of which may not always be necessary.

B. Single-Barrel Siphons

Single barrel siphons can be used for conveying stormwater flows where there are periods of no flow during which maintenance can be provided.

Even though some agencies limit the slope of the rising leg of the siphon to 15%, steeper slopes and even vertical drops and risers are acceptable, if maintenance chambers with debris collection sumps at the bottom are provided at the drop and riser of the siphon, as shown in Figure 9.32.

Sloping legs of siphons (Figure 9.33) have been designed without maintenance chambers, however, the chambers provide maintenance flexibility with direct access to service the siphon. The steeper the legs of the siphon, the more difficult it is to clean the siphon from shallow manholes located near the ground surface, and deep maintenance chambers reaching to barrel inverts may be required.

Where a vertical drop and riser are provided, they should serve as maintenance chambers and include access down to the barrels and sumps. Sumps located at the bottom of the maintenance chambers trap the debris that accumulates in the siphon.

C. Multi-Barrel Siphons

In channels or sewers that convey a continuous flow, where one barrel does not have sufficient capacity and the flow has to be divided,

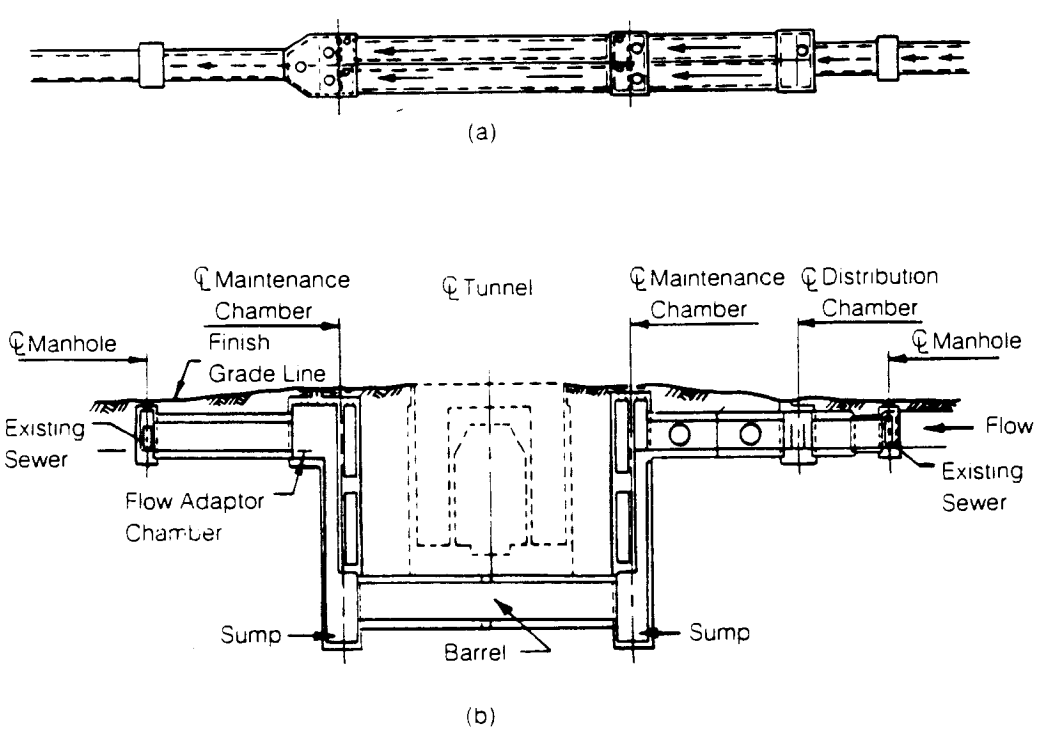

Figure 9.32—Profile and plan of double barrel siphon vertical legs: (a) plan and (b) siphon profile (Engineering News, 1916).

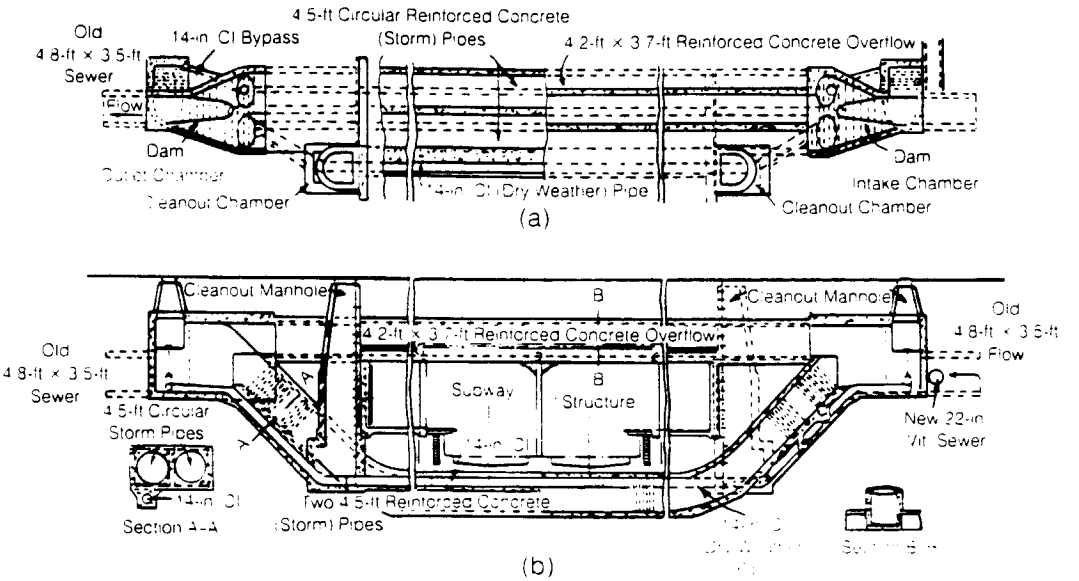

Figure 9.33—Profile and plan of a double barrel siphon sloping legs: (a) sectional plan and (b) longitudinal section (in. × 25.4 = mm and ft × 0.304 8 = m) (Engineering News, 1916).

or where redundancy is required by local agencies, the multi-barrel siphon is applicable. Plan and profiles of such a siphon are shown in Figure 9.33.

Where redundancy is required for maintenance purposes, one additional equal capacity barrel is sufficient. To fulfil its functions, the multi-barrel siphon requires special equipment and structures, including gates that close the barrel to be maintained while the other barrel is open.

Special structures may also include a flow distribution chamber and a flow adaptor chamber. These chambers are used to contract and expand the flows. The distribution chamber serves to direct the flow from one sewer to the two barrels of the siphon alternatively used, while the flow adaptor chamber serves to direct the flow from the two barrels of the siphon to one conduit.

D. Design Criteria and Practices

One of the critical criteria for the design of siphons is the maintenance of self-cleansing velocities under widely varying flow conditions (ASCE 1969). Siphons used for conveying storm water are usually designed for a flow velocity of 3 fps for a 5-year return interval design flow. Siphons with water containing abrasive suspended materials should be designed for a flow velocity less than 10 fps.

The head losses through each of the siphon components must be estimated for the purpose of plotting the hydraulic grade line. Upstream surcharging should be avoided, and therefore one of the main design objectives should be to minimize the head losses through the siphon. The friction losses can be estimated by using the combined Darcy-

Weisbach and Manning's Equation. The equation is useful in the following form:

$$h_f = \frac{29.1n^2 LV^2}{r^{4/3}\, 2g} \tag{9-16a}$$

and, in metric units:

$$h_f = \frac{19.5n^2 LV^2}{r^{4/3}\, 2g} \tag{9-16b}$$

where:

h_f = head loss (ft)
n = Manning's friction factor
L = length of conduit (ft)
r = hydraulic radius (ft)
V = velocity (fps)
g = acceleration of gravity (ft/sec^2)

Minor losses such as bend, contraction and expansion, and entrance and exit losses can be estimated using tables and charts appearing in several publications, such as Brater and King (1976) and City of Baltimore (1972). These losses are also discussed in Chapter 6. It should be noted that head losses in siphons can be significant, particularly in flat coastal areas, where the low terrain does not allow for surcharge and the available project corridor is narrow.

The size of the barrel or conduit can be determined initially based on the minimum required flow velocity. However, the barrel can be sized accurately only after the hydraulic losses are estimated. If the head loss under the design flow condition is excessive, an increase in size of the conduit should be considered.

VIII. SIDE-OVERFLOW WEIRS

A. Definitions and Applications

Side-overflow weirs facilitate overflow and diversion of stormwater by directing the discharges away from the original channel. Such structures are commonly used in combined sewers where the weirs permit the combined flows to build up to a design level, at which time overflow is initiated and directed from the side weir into relief sewers or natural water courses. Since the design overflow level is located above the level of the sanitary flow, the dry-weather sanitary flow is discharged continuously toward sewage treatment facilities. Flow diversions occur only during storms (Figure 9.34). Side overflow weirs are also used to direct channel discharges above predetermined levels into off-line stormwater detention facilities.

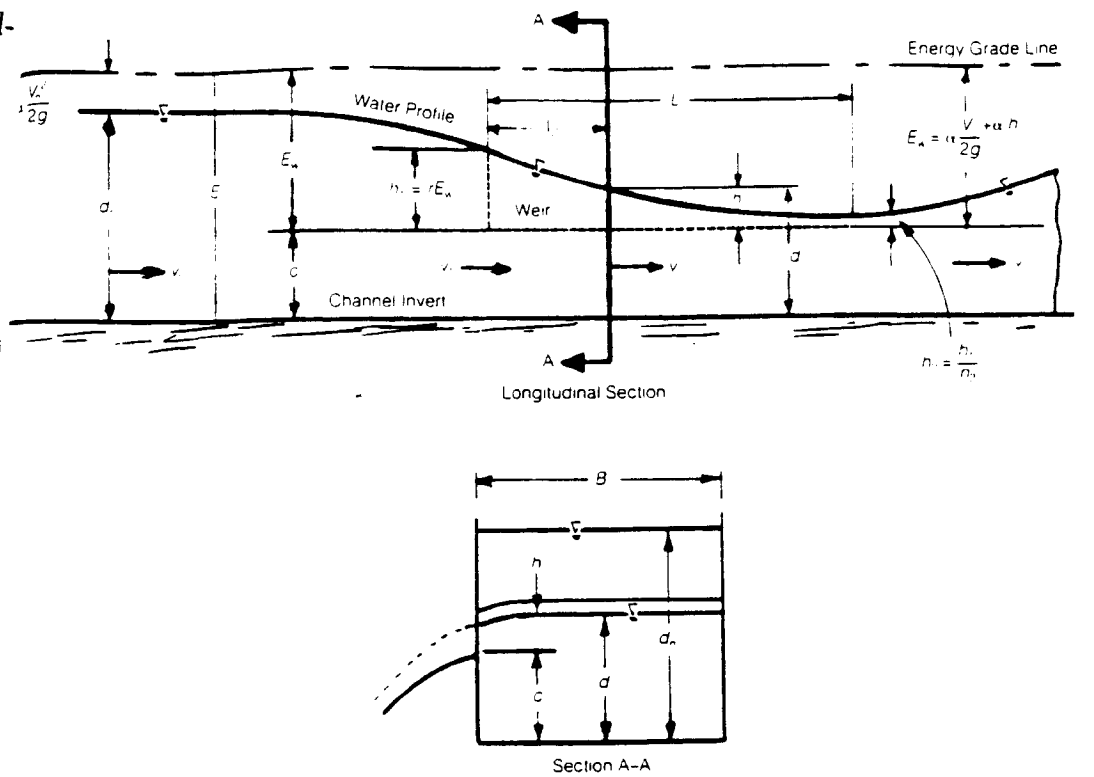

Figure 9.34—Typical cross sections at a side weir (Metcalf & Eddy, 1972).

B. Design Considerations

The design of side weirs is based on empirical equations which quantify the relationship between the discharge over the weir and geometric parameters at the weir, including the length of the weir and head (Hager 1987). Figure 9.35 (Metcalf and Eddy 1972) shows three head or water-surface profile conditions that can prevail at a side weir. These conditions were defined by DeMarchi in 1934 (Collinge 1957):

(a) Condition 1: The channel bed slopes steeply, producing supercritical flow. Under this condition the weir has no effect upstream and along the weir there is a gradual reduction in depth. Downstream of the weir the flow depth in the original channel increases, tending asymptotically to the normal depth corresponding to the remaining discharge.
(b) Condition 2: The channel bed slopes mildly. Under this condition subcritical flow prevails and the weir impact is noticed upstream of the weir only. The water surface profile downstream of the weir corresponds to the normal depth of the remaining discharge. Along the weir there is a gradual increase in depth and upstream of the weir the flow depth tends asymptotically to the normal depth for the initial discharge.
(c) Condition 3: The channel bed slopes mildly, but the weir crest is below critical depth corresponding to the initial flow, and the flow at the weir is supercritical. Recent studies (Frazer 1957) indicate that conditions 1 and 3 may result in development of a hydraulic jump at the weir.

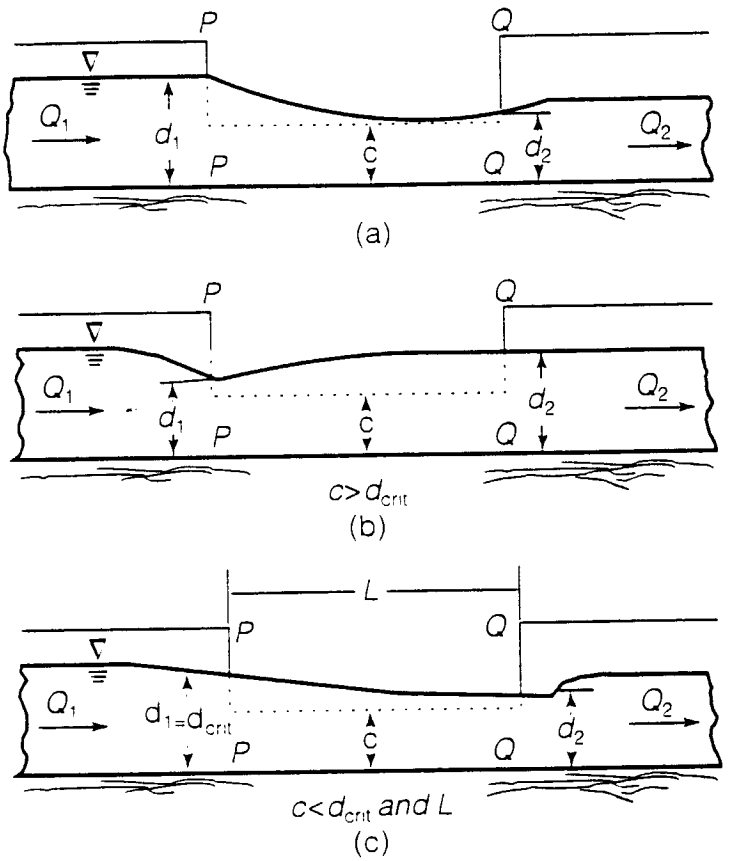

Figure 9.35—Possible types of water surface profiles at a side weir (a) condition 1, (b) condition 2, and (c) condition 3 (Metcalf & Eddy, 1972).

The most common condition that a designer will encounter is Condition 3, where the weir elevation is below the critical depth. When only a relatively small amount of the flow is diverted, a rising water surface profile occurs. According to Metcalf and Eddy Inc. (1972), the falling profile results if the ratio of the height of the weir (c) to the channel specific energy (E_w) referenced to the top of the weir is less than 0.6.

C. Design Practices

1. *Falling Water Surface*

The equations and procedures for computing weir length for the falling water surface profile were developed by Ackers (Chow 1959). These equations combine Bernoulli's theorem with a weir discharge formula. Metcalf and Eddy Inc. (1972) suggest using:

$$L = 2.03B\left(5.28 - 2.63)\frac{c}{E_w}\right) \tag{9-17}$$

Where:

L = length of weir
B = channel width
c = height of weir
E_w = channel specific energy

and,

$$E_w = \alpha\frac{v^2}{2g} + \alpha'(d_n - c) \tag{9-18}$$

where:

α = velocity coefficient
V = normal velocity in the approach channel, fps
g = acceleration due to gravity, ft/sec^2
α' = pressure-head correction
d_n = normal depth of flow in approach channel, ft.
c = height of weir above channel bottom, ft.

α and α' of 1.2 and 1.0, respectively, can be used in the approach channel, while at the lower end of the weir values of 1.4 and 0.95 can be used for α and α', respectively.

2. *Rising Water Surface*

The analysis for estimating the weir length for rising water surface is based on the theoretical equations developed by DeMarchi (Collinge 1957):

$$L = \frac{B}{C}\left[\phi\left(\frac{d_2}{E}\right) - \phi\left(\frac{d_1}{E}\right)\right] \tag{9-19}$$

where:

L = length of weir, ft.
B = channel width, ft.
C = constant (0.35 for a free nappe).
$\phi(d/_E)$ = varied flow function taken from Fig. 3 in Collinge (1957).
d_1, d_2 = depths in channel, ft.
E = specific energy, ft.

This equation is recommended for use only in case of a rising water surface. Metcalf and Eddy Inc. (1972) indicates that this equation works best when the Froude number is between ± 0.3–0.92.

IX. FLOW SPLITTERS, JUNCTIONS, FLAP GATES, AND MANHOLES

A. Flow Splitters

A flow splitter is a special structure designed to divide a single flow and divert the parts into two or more downstream channels. A flow splitter can serve two functions:

(a) Reduction in water surface elevation—By dividing the flow from a large pipe into multiple conduits, the height of flow (measured from the flow line to the water surface (or for pipes flowing full, the inside diameter) can be reduced. This may be necessary to route flows under immovable obstructions.
(b) Dividing flows wherever necessary—Examples of this include division of existing large special-design conduits, such as arches or horseshoes, into less expensive multiple-pipe continuations, and division of flow between low- and high-flow conduits at the intake of an inverted siphon.

Two major considerations exist for the design of flow-splitting devices:

(a) Head Loss—Hydraulic disturbances at the point of flow division result in unavoidable head losses. These losses, however, may be reduced by the inclusion of proper flow deflectors in the design of the structure. Deflectors minimize flow separation by providing a gradual transition for the flow, rather than by forcing abrupt changes in flow direction. (See also the discussion in Chapter 8.)
(b) Debris—In all transitions from large to smaller pipes, debris accumulation is a potential problem. Tree limbs and other debris that flow freely in the larger pipe may not fit in the smaller pipe(s) and may restrict flow. In addition, flow splitters cause major flow disturbances resulting in regions of decreased velocity. This reduction causes material suspended in the stormwater flow to settle in the splitter box. Although the deflector design should minimize velocity reduction as much as possible, total elimination of the problem is unlikely. Therefore, positive maintenance access must be provided. Because flow splitting devices are maintenance-intensive, their use should be judiciously controlled by the engineer.

B. Junctions

A junction is a region of converging or intersecting flow occurring in either closed-conduit or open-channel systems. Since one or more of the incident flows must undergo a change in velocity, energy losses are experienced at junctions. Junction losses are also discussed in Chapters 6 and 8. For further information on closed-conduit junctions in storm drainage systems, the engineer is referred to the Los Angeles Manual on the Hydraulic Analysis of Junctions (City of Los Angeles 1968), Street & Highway Drainage, Volume 2, Design Charts (University

of California 1965) and the Denver Urban Storm Drainage Criteria Manual, (Urban Drainage and Flood Control District 1984).

C. Flap Gates

Flap gates, usually made of cast iron or cast steel, are used to permit flow in only one direction. A small differential pressure on the back of the gate will open it, allowing discharge in the desired direction. When water on the front side of the gate rises above that on the back side, the gate closes to prevent backflow. The seat or ring of the flap gate is attached to a headwall or directly to the pipe that forms the opening through which flow passes. Larger flap gates are often mounted to a cast iron thimble, cast in the headwall or other structure through which flow is to pass. Flap gates are available for round, square, and rectangular openings and in various designs.

Flap gates can act as a skimmer and cause brush and trash to collect between the flap and the seat at low flow. Rubber flap gates (Brombach 1990) and "duck bill" (Freeman et al. 1990), which are less susceptible to this sort of clogging, are also being used. Periodic inspection and cleaning should be scheduled when the water flowing through the gate carries floating material. If the gate is to be kept clear of debris, it should be mounted 12–18 inches (30–46 centimeters) above the apron in front of the gate. This allows room at the bottom for floating material to work its way through the gate. When this clearance is not available, more frequent inspection and cleaning must be provided.

For those drainage structures that have a flap gate mounted on a pipe projecting into a stream, the gate must be protected from damage by floating logs or ice during high flows. In these instances, protection must be provided on the upstream side of the gate.

D. Manholes

The primary function of a manhole is to provide convenient access for inspection, maintenance, and repair of storm drainage systems. Secondary functions include provision for multiple pipe intersections, ventilation, and pressure relief. Manhole design is covered in Chapters 6 and 8.

1. Size

On small lines, a minimum inside diameter of 4′ (see Figure 9.34), tapering to a cast-iron frame, that provides a clear opening usually specified as 24″, has been widely adopted. Immediately under the frame, the diameter should be at least 24″, generally enlarging to 30″ within 2′ of the surface, but some authorities allow a diameter of 24″ for a distance of 24″ below the frame. Occasionally, the working space of a manhole is a rectangular vault, but the opening and access usually remain circular.

It is common practice to use eccentric cones, especially in precast manholes, to provide a vertical side for the steps and to avoid interference with street curbs. Most often the orientation places the steps on the wall over the bench, but some designs place the steps on the wall opposite the outlet pipe.

Instead of extending the 4-foot diameter section up to a cone near the surface, some engineers prefer a design that maintains the 4-foot section to a suitable working height, tapering then to 3′ as shown in Figure 9.36. The cast iron frame in this case has a broad base to rest on the 36″ diameter shaft. Still another design uses a removable flat reinforced concrete slab instead of a cone, as shown in Figure 9.36(d). This is applicable whether the working space is circular or rectangular. The slab must be suitably reinforced to withstand traffic loads.

2. *Frame and Cover*

The manhole frame and cover are normally made of cast iron. The cover is designed to provide a good fit between cover and frame, and with adequate strength to support superimposed loads. The closure should be relatively tight, and the cover should be designed to prevent unauthorized entry. It is a good practice to identify manholes with covers with the words "Storm Sewer" or equivalent cast into the top surface to differentiate them from those on sanitary sewers, communication conduits, or other underground accessways.

3. *Steps*

It is common practice in many areas to abandon the use of manhole steps in favor of having maintenance personnel supply their own ladders. Reasons for this include danger from rust-damaged steps, ease of access to children, and debris snagging. Steps coated with neoprene or epoxy or steps fabricated from a rust-resistant metal such as stainless steel or aluminum are preferable to steps made from reinforcing steel.

4. *Manholes on Pipes Flowing Full*

It often becomes necessary to design a storm drainage conduit to flow full. This poses special problems to the designer because of the need to minimize head loss (see Chapters 6 and 8 for a more complete discussion). If the hydraulic grade line is designed to rise above the ground surface at a manhole site, special consideration must be given to the design of the frame and cover. The cover must be secured so that it remains in placing during peak flooding periods, avoiding a manhole "blow-out." This may be accomplished with a bolted manhole cover or by use of a cover with a locking mechanism. Leakage from around the edges of such manhole covers is usually minimal and not of concern to the designer.

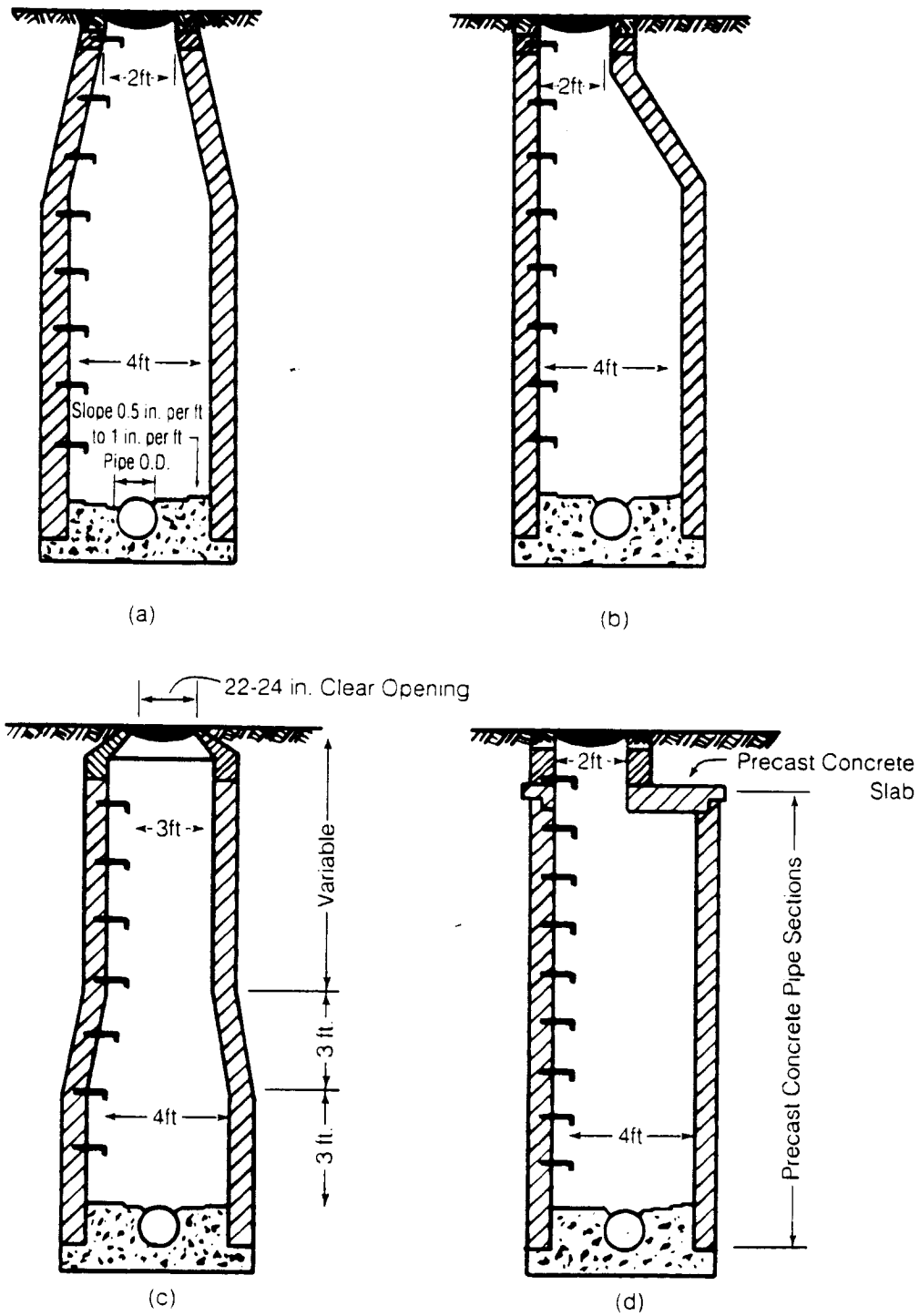

Figure 9.36—Typical manholes for small sewers: (a) constant diameter tapering to the frame, (b) eccentric cones, (c) 4-ft working section tapering to 3-ft section, and (d) removable flat reinforced concrete slab (ft × 0.304 8 = m, in. × 2.54 = cm).

5. *Deep Manholes*

Deep manholes must be carefully designed to withstand soil pressure loads. If the manhole is to extend very far below the water table, it must also be designed to withstand the associated hydrostatic pressure (see Chapter 14) or excessive seepage may occur. Access must be provided, and very deep manholes must be supplied with either steps or built-in ladders, because long ladders would be cumbersome and dangerous in deep manholes.

X. STORMWATER PUMPING

A. Introduction

Stormwater pumping is generally used only when gravity drainage is not feasible, although there may be cases where it is more economical to pump than to construct gravity conveyance systems. Storage in storm drains, wet wells, or ponds can reduce the pumping capacity required to handle peak runoff rates. The following is adapted from the "Manual for Highway Stormwater Pumping Stations" (USDOT 1982), which, along with other publications, should be consulted for the final design of stormwater pump stations.

B. Planning and Site Considerations

Several important considerations affect planning and site selection for pump stations. The access necessary for safe operation, maintenance, and emergency functions must be available at all times. Hydraulic conditions will have primary importance in site selection, but site appearance and sound attenuation should also be assessed. Foundation investigations are essential (although they rarely reveal conditions unstable enough to merit rejection of a site), and the effects of soil erosion and contaminated runoff on the pump station influent should be evaluated, as well as the rate of discharge and water quality of the effluent. Enough space must be provided in the area outside the station to accommodate parking as well as movements of large machinery.

A dependable energy source is essential. The primary source of electrical power for most stormwater pump stations is a public utility. Underground service is preferred for safety and aesthetic reasons, and overhead lines into the station should be avoided, as they present potential safety hazards during large equipment operation.

C. Design Features

Some features to evaluate when designing a pump station depend on the size of the facility, while others relate to site conditions, economic considerations, and operating needs. Table 9.9 lists these features and

rates the parameters of each in terms of high, medium, and low conditions. A stormwater pump station selection matrix is presented in the USDOT manual (1982).

D. Storage

Stormwater storage can reduce the required peak capacity of a pump station. Selection of a peak pumping capacity is a trial and error process that considers the inflow hydrograph, available storage, and possible pump discharge rates. An approximate method developed by Baumgardner (1981) yields results that can be evaluated during selection of final design conditions using computer program developed by the FHWA.

The approximate method includes adjusting the inflow hydrograph (developed using appropriate procedures presented in Chapter 5 of the manual) to an equivalent triangular hydrograph, as shown in Figure 9.37. An estimate of the storage required to reduce the peak pumping rate to a desired value can be found by assigning a peak pumping rate and plotting it as a horizontal line, as shown in Figure 9.38 (U.S. D.O.T. 1982).

The area of the triangular hydrograph above the peak pumping rate represents an estimate of the storage volume required. This assumes that storage below the last pump-on elevation will not affect the design. The effect of this storage on final design can be considered using the FHWA computer program, or a mass curve routing procedure as presented in Section F. The FHWA program is currently available only for microcomputer applications. The required storage can be estimated by the equation:

$$\frac{V_s}{V_t} = \left(\frac{\Delta Q}{Q_p}\right)^2 \tag{9-20}$$

where:

V_s = Required storage volume, ft^3
V_t = Volume of triangular inflow hydrograph, ft^3
ΔQ = Peak flow reduction, cfs
Q_p = Peak flow of triangular inflow hydrograph, cfs

A graphical presentation of the relationship in Equation 9-20 is shown in Figure 9.39. By selecting a peak reduction ratio ($\Delta Q/Q_p$), the storage ratio (V_s/V_t) can be obtained directly. When the inflow hydrograph volume (V_t) is known, the storage required is estimated as the product of the storage ratio and the inflow hydrograph volume.

E. Pumps

Pumps are commonly used in storm drainage systems, though the designer is cautioned that they should be used only when gravity flow is not possible.

TABLE 9.9. Pump Station Design Features and Considerations (U.S. DOT 1982).

Feature or Consideration	High Condition	Medium Condition	Low Condition
1. Station Design Capacity	Maximum exceeding 300 ft^3/sec	Maximum between 100 and 300 ft^3/sec	Maximum less than 100 ft^3/sec
2. Station Design Head	Over 35′ TDH	Between 15′ and 35′ TDH	Less than 15′ TDH
3. Storage Upstream of Pumps	For velocity reduction, settlement of solids, minimizing equipment	Used if available	Not required or available
4. Quality of Pumped Water	Turbid and sand-laden inflow	Moderate contamination	Minimal contamination
5. Inflow Rate	Rapid increase	Normal hydrograph	Slow increase
6. Discharge Conditions	Long rising outfall from each pump	Short free outfall from each pump	Limitation of discharge rate
7. Sump Dewatering	Sump pump required	Vacuum truck preferred	No provision
8. Electric Power Reliability	Completely dependable; dual service	Very dependable; single service	Undependable; frequent outages
9. Natural Gas/LPG Desired as Fuel	Completely dependable; dual service	Very dependable; good storage	Not readily available; supply unreliable
10. Station Siting	Good access from frontage road or similar	Good access from highway	Poor access, alongside highway
11. Soil Conditions	Rock	Hard, steep unshored cuts	Clay or soft soil
12. Foundation Conditions	Acceptable bearing strata	Piling required for bearing	Extensive dewatering with piling required because of uplift
13. Above-Ground Structure	Large acceptable	Moderate preferred	Smallest possible desired
14. Structure Visibility	Large structure acceptable	Modest structure desired	Minimum only acceptable

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15. Initial Cost	High capital cost acceptable	Moderate cost acceptable	Lowest cost mandatory
16. Maintenance Capability	Excellent, with complex machinery	Reasonably good	Mediocre
17. Operating Cost	High cost acceptable	Moderate budget desired	Lowest budget desired
18. Equipment Handling Devices (Built-in)	Elaborate type considered essential	Single type acceptable	Minimum or none required
19. Equipment Handling Devices (Mobile)	Use preferred for all requirements	Used to supplement built-in	Not required due to elaborate built-in
20. Trash Handling Devices	Elaborate built-ins preferred	Simple built-in adequate	Vacuum trucks preferred
21. Pre-Screen Inflow Debris	—	—	—
22. Hazardous Spill Vulnerability	—	—	—
23. Epoxy Coatings and Lining of Pumps	—	—	—
24. Grease Lubrication for Pumps	—	—	—
25. Steelwork Galvanizing	—	—	—
26. Manifold to Pressure Discharge	—	—	—
27. Sediment and Hydrocarbon Removal from Discharge	—	—	—
28. Emergency Generator	—	—	—
29. Supervisory Control (Telemetering)	—	—	—

(USDOT 1982).

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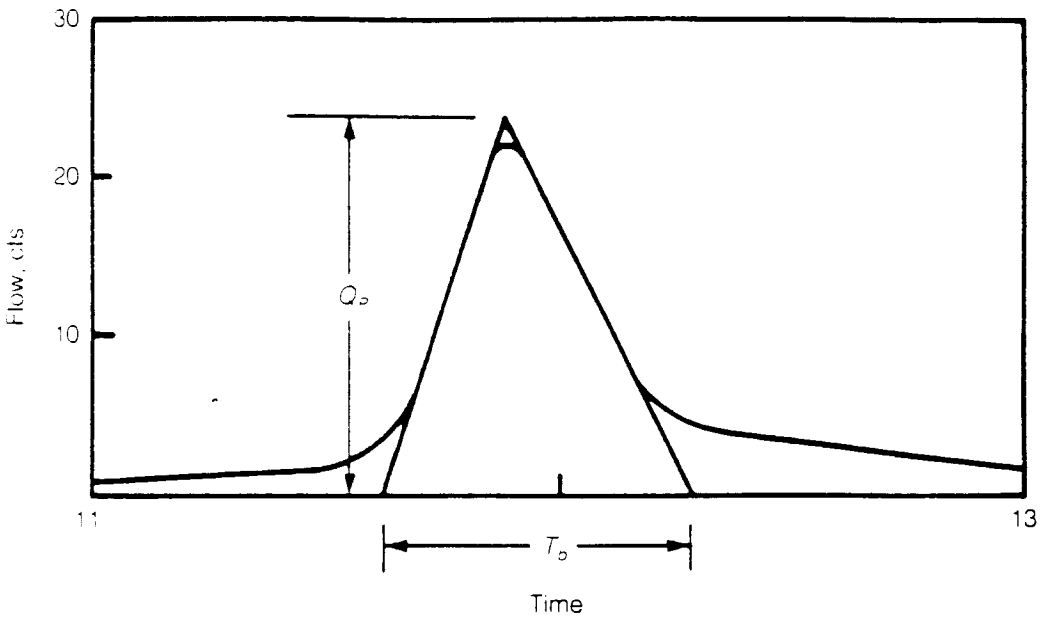

Figure 9.37—Triangular approximation of an inflow hydrograph (cfs × 0.028 32 = m³/s) (Fed. Highw. Adm., 1982).

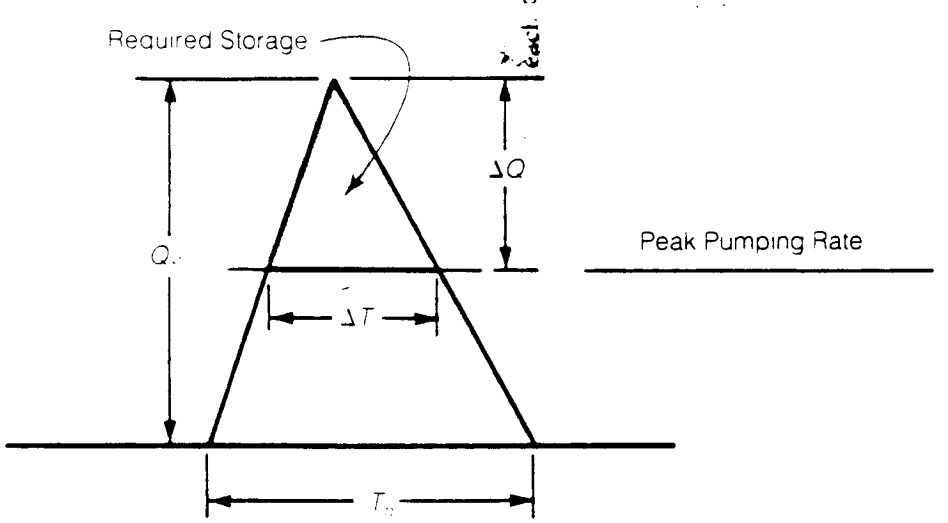

Figure 9.38—Estimation of required storage based on a selected peak pumping rate (Fed. Highw. Adm., 1982).

1. *Pump Types*

Stormwater pumping typically uses large pumps that operate at relatively low heads. The main types of pumps include:

(a) Vertical (propeller and mixed flow)
(b) Submersible (vertical and horizontal)
(c) Centrifugal (horizontal non-clog)
(d) Screw
(e) Volute or Angleflow (vertical)

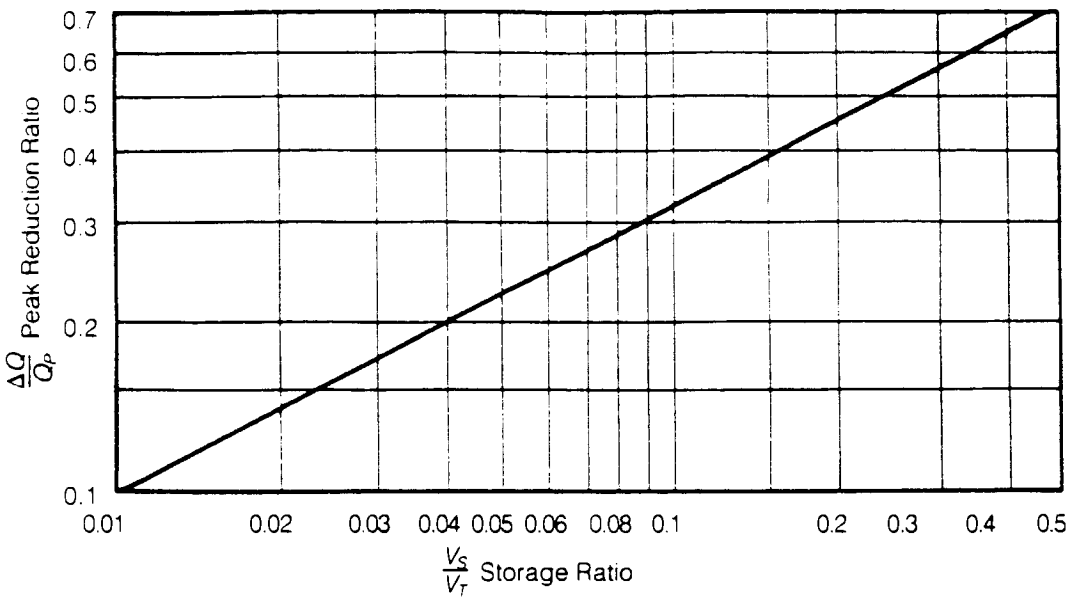

Figure 9.39—Relationship between peak reduction ratio and storage ratio: $V_S/V_T = (\Delta Q/Q_P)^2$ *(Baumgardner, 1981).*

All of these pumps generally are driven by electric motors, but diesel or gas engines are sometimes used for vertical, screw, or angleflow pumps.

Vertical Pumps Vertical pumps, the type used most frequently in stormwater pump stations, are available in many capacities and discharge heads. Single-stage propeller pumps are used for low heads; mixed-flow pumps for higher heads. Two-stage propeller pumps are available that will approximately double the head capacity of a single-stage pump. A typical vertical stormwater pump is shown in Figure 9.40.

Submersible Pumps Until recently, submersible pumps, except at small stations, have been mainly used as accessory pumps to handle low (dry weather) flows, groundwater infiltration, and cleanup and pumpout of the pump pit after a storm. This has been changing as larger submersible pumps have become available, and large stormwater pump stations can now be designed that rely solely on submersible pumps.

Centrifugal Pumps A special type of centrifugal pump is used in dry-pit stations. Designed to handle suspended solids and debris which pass the screens, these pumps are the end suction, non-clog type. Enclosed impellers and a hand-hole on the casing permit inspection and removal of foreign material. Small centrifugal pumps combined with upstream storage are well suited to small catchments.

Screw Pumps Screw pumps are appropriate when pressurized transfer is not required. They are typically used in agricultural areas and offer

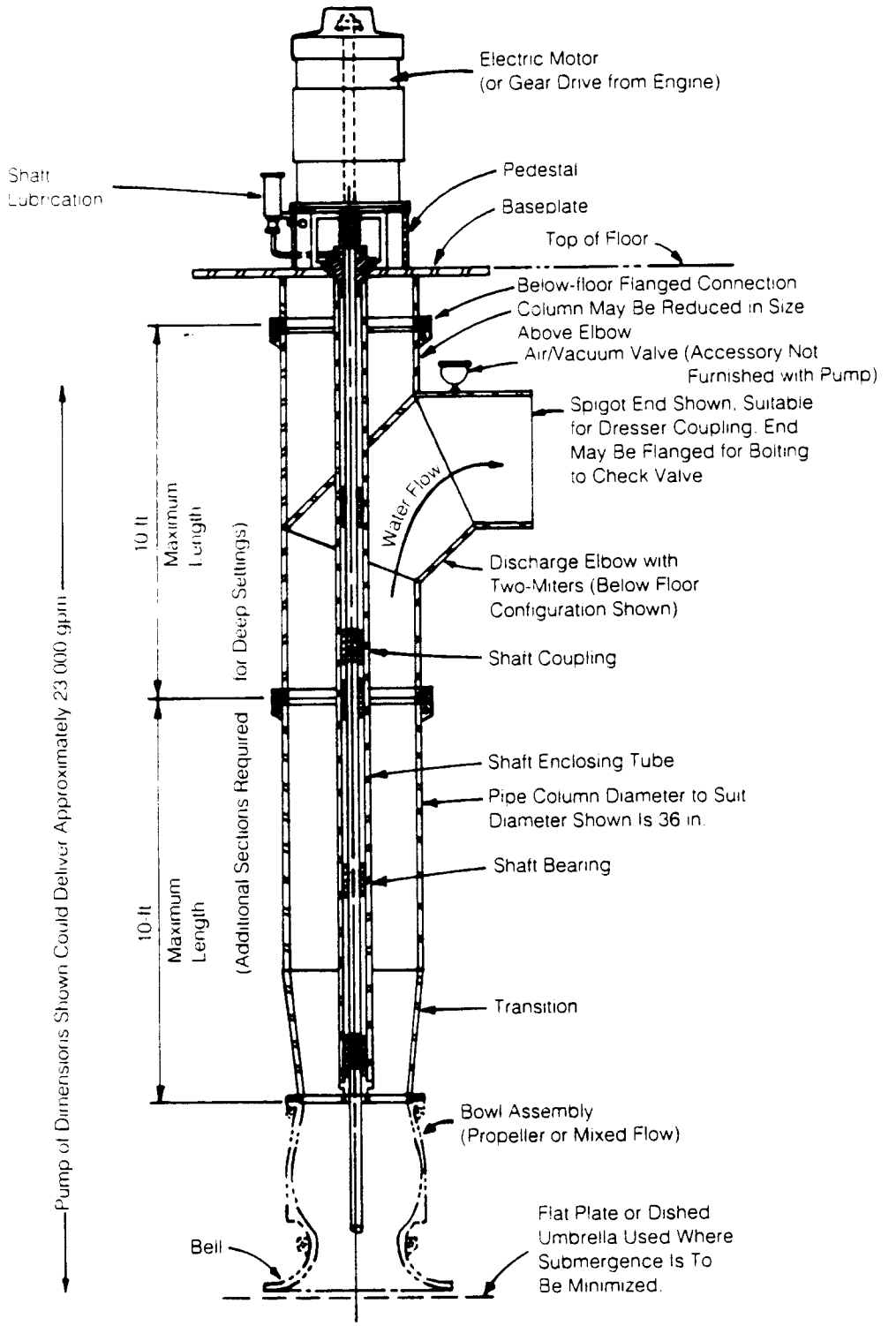

Figure 9.40—Typical vertical stormwater pump (oil-lubricated enclosed shaft shown) (gpm × 0.063 08 = L/s and ft × 0.304 8 = m) (Fed. Highw. Adm., 1982).

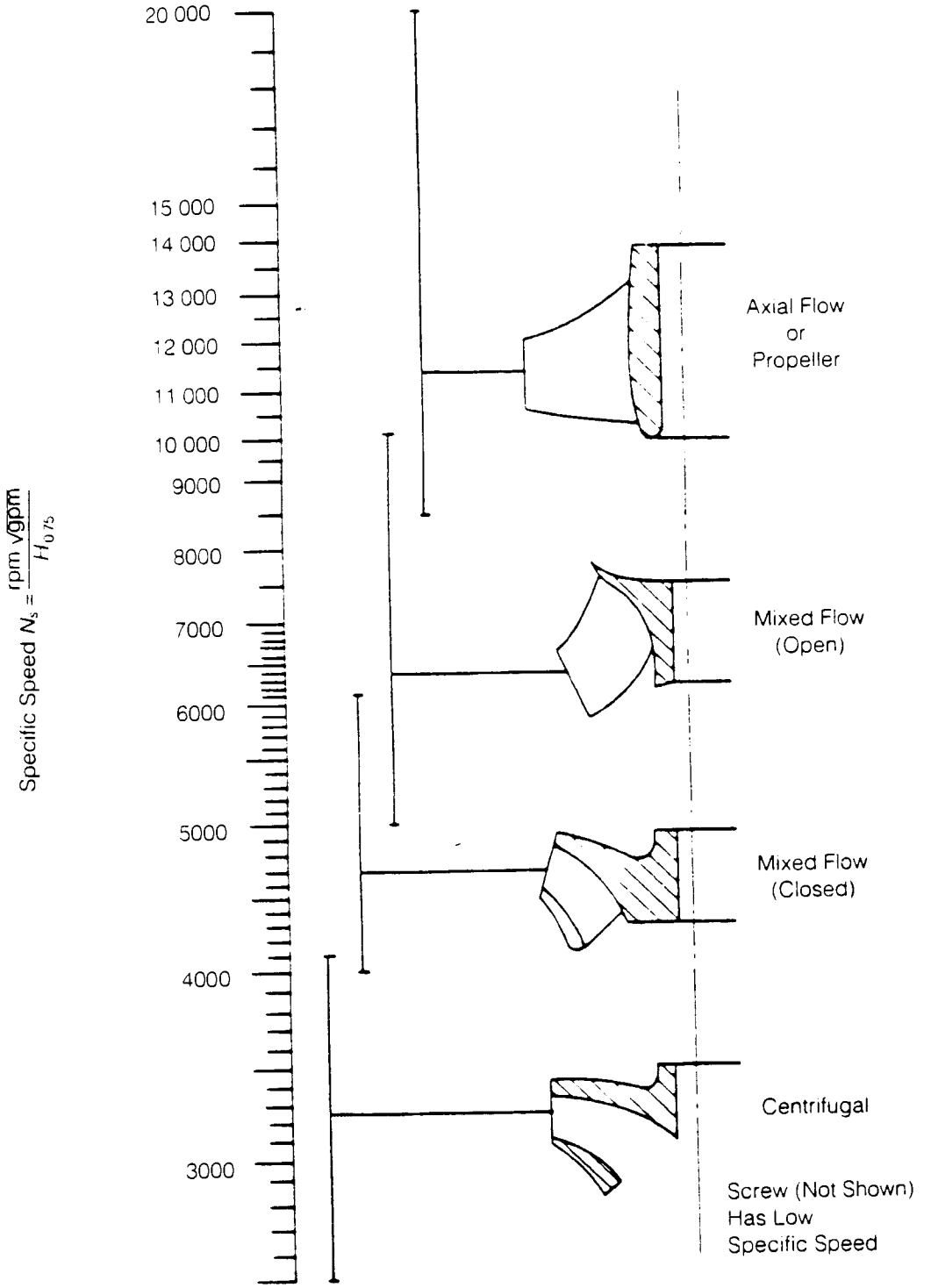

Figure 9.41—Specific pump speed versus impeller types (gpm × 0.063 08 = L/s) (Fed. Highw. Adm., 1982).

a simple and reliable method of stormwater pumping when site conditions are suitable.

Volute or Angleflow Pumps The volute or angleflow pump, also called a dry-pit angleflow pump or a single-suction centrifugal mixed-flow pump, is usually mounted in a vertical position, with the motor above the pump room operating floor and the pump as much as 25′ below. Vertically mounted volute or angleflow pumps have been a standard in sewage pump station installations for many years. However, because of the complex accessories needed with these pumps, they are not recommended for stormwater pumping unless the level of operating head makes it necessary.

2. *Pump Selection*

A pump characteristic known as specific speed should be considered during pump selection. Pump impellers for high heads have low specific speeds, while impellers for low heads usually have high specific speeds. Figure 9.41 (U.S. D.O.T. 1982) shows the relationship of different types of impellers to the useful range of specific speeds.

Pumps of the axial-flow type have impellers shaped like ship propellers, and as the name implies, the liquid is discharged axially through the impeller. Pumps of the mixed-flow type have impellers with vanes integral with a conical hub. The pumping head is developed partly by centrifugal force and partly by a lifting action of the vanes. The application of the mixed-flow pump is similar to that of the vertical axial-flow pump. Pumps of the centrifugal type have impellers that develop head entirely by centrifugal force.

The performance curve developed by the manufacturer should be obtained before selecting a particular pump. Procedures on pump selection are presented in a Federal Highway Administration manual (U.S. Department of Transportation 1982). The advantage of using standard equipment, however, often outweighs the need to obtain a perfectly tailored design.

F. Mass Curve Routing

A wide range of storage and pumping rate combinations should be evaluated when considering pumping cycles for frequent small volume storms and the potential for flooding from less frequent, large-volume storms. Usually, computer programs are best suited to this type of trial and error evaluation. A mass curve routing procedure developed by Baumgardner (1982) follows.

(a) Develop an inflow hydrograph, a stage-storage curve, and a stage-discharge curve for the range of pumping facilities being considered. If the pumping rate is constrained by downstream or environmental considerations, the required storage must be determined by various trials of the mass curve routing procedure.

(b) Establish the point at which the cumulative flow curve has reached the storage volume associated with the first pump-on elevation.
(c) Draw the pump discharge line from the intersection point located in Step 2, upward to the right at a slope equal to the discharge rate of the pump. The vertical distance between the mass inflow curve and the pump discharge curve represents the amount of stormwater which must be held in storage.
(d) To determine the maximum storage volume required, a line is drawn parallel to the pump discharge curve, and tangent to the mass inflow curve. The vertical distance between this tangent and the pump discharge line represents the maximum storage volume required.
(e) The storage pipe and wet well should be designed to handle sediment, while the pumping system should be designed to safely carry sediment which is flushed from the wet well.

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CHAPTER 10

COMBINED SEWER SYSTEMS

I. GENERAL DESCRIPTION OF COMBINED SEWERAGE SYSTEMS

A. Introduction

A combined sewer consists of a single conduit that collects and transports domestic sewage and industrial wastewater, along with stormwater runoff. The dry weather flow, which consists of domestic sewage and industrial wastewater as well as groundwater infiltration, is intercepted and conveyed to a sewage treatment plant for processing. During wet weather (events such as a rainstorm or snowmelt), the combined stormwater runoff and sewage usually exceeds the capacity of the interceptor and the treatment plant and overflows through an outfall into receiving waters with minimum, or no, treatment. Because of their large volumes, these combined sewage overflows (CSOs) are a major source of pollution during and following wet weather events. A typical layout of an urban combined sewer system is shown on Figure 10.1 (Kibler 1982). In a combined sewer system, the maximum stormwater runoff typically exceeds dry-weather flow by 50–200 or more times. Therefore, combined sewers are designed to convey stormwater runoff, that is, as storm sewers.

While this chapter provides an introduction to combined sewer systems, the reader is encouraged to consult the excellent recent Manual of Practice entitled "Combined Sewer Overflow Pollution Abatement" (WPCF 1989) for more detailed information.

B. Flow Regulation Devices

1. Purpose

Combined sewers are typically designed to carry 2–3 or more times dry-weather flows (though in some cases they cannot even carry all of the dry-weather flows). Wet-weather flows are typically much greater

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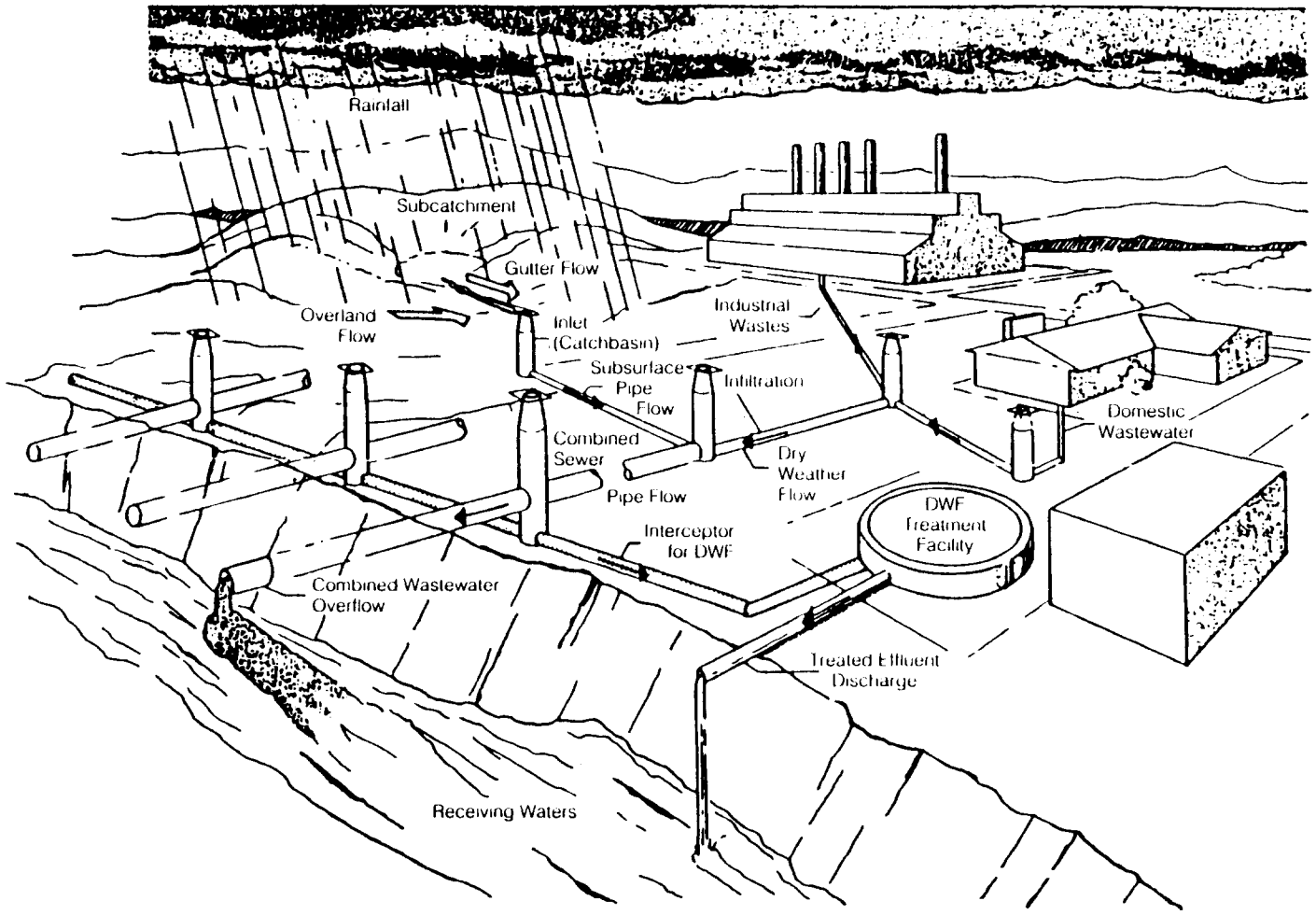

Figure 10.1—Typical urban combined sewer system (Kibler, 1982).

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than that, and regulators are used to control flows in the system. The combined sewer flow regulator is designed to:

(a) Divert dry-weather flows from tributary combined sewers to the interceptor for conveyance to the treatment plant.
(b) Direct excess flows to CSOs (or to CSO storage/treatment devices).
(c) Minimize the extent and frequency of combined sewage discharges into receiving waters by allowing overflows at selected locations, and by using the potential storage of the tributary sewer system.
(d) Prevent surcharging of the intercepting sewer.
(e) Prevent entry of debris into the intercepting sewers.

The regulators are of two types: static and dynamic. The static regulator, as the name implies, has no moving parts and must be adjusted manually. Flow through these devices increases with increases in upstream head. A dynamic regulator, on the other hand, functions semi-automatically or automatically, responding to the water level in the combined sewer to limit the flow diverted to the interceptor. The Water Pollution Control Federation (1989) Manual of Practice describes a variety of regulators.

2. *Static Regulators*

Static flow regulators are of many types, several of which are described herein. It should be noted that most static regulators operate efficiently over a limited range of flows, and care should be exercised in their selection.

Fixed Orifice This regulator consists of a small dam constructed across the combined sewer, with a vertical orifice just upstream of the dam. The purpose of the dam is to divert the dry weather flow through the vertical orifice to the interceptor (Figure 10.2). During wet weather, the flow exceeding the capacity of the orifice flows over the dam and through the outfall to the receiving waters, or to CSO storage/treatment. The orifice is either circular or rectangular. The size of the opening is designed to intercept peak dry weather flow; however, the flow diverted to the interceptor during wet weather is higher. This type of regulator, generally used for flows less than 2 cfs, is not recommended because of its tendency to plug up with debris, and because the orifice discharge varies widely with head.

Leaping Weir This is an opening located at the invert of the combined sewer. The opening is provided with an adjustable plate having a raised lip on the upstream side as shown on Figure 10.3. The opening of the leaping weir is designed to divert dry-weather flows from the combined sewer to the interceptor, or to CSO storage/treatment. During wet weather, the excess flow leaps over the weir for discharge through the outfall. To improve the interception of dry weather flow, the downstream end of the orifice is tipped up. The leaping weir is typically used for flows up to 4 cfs.

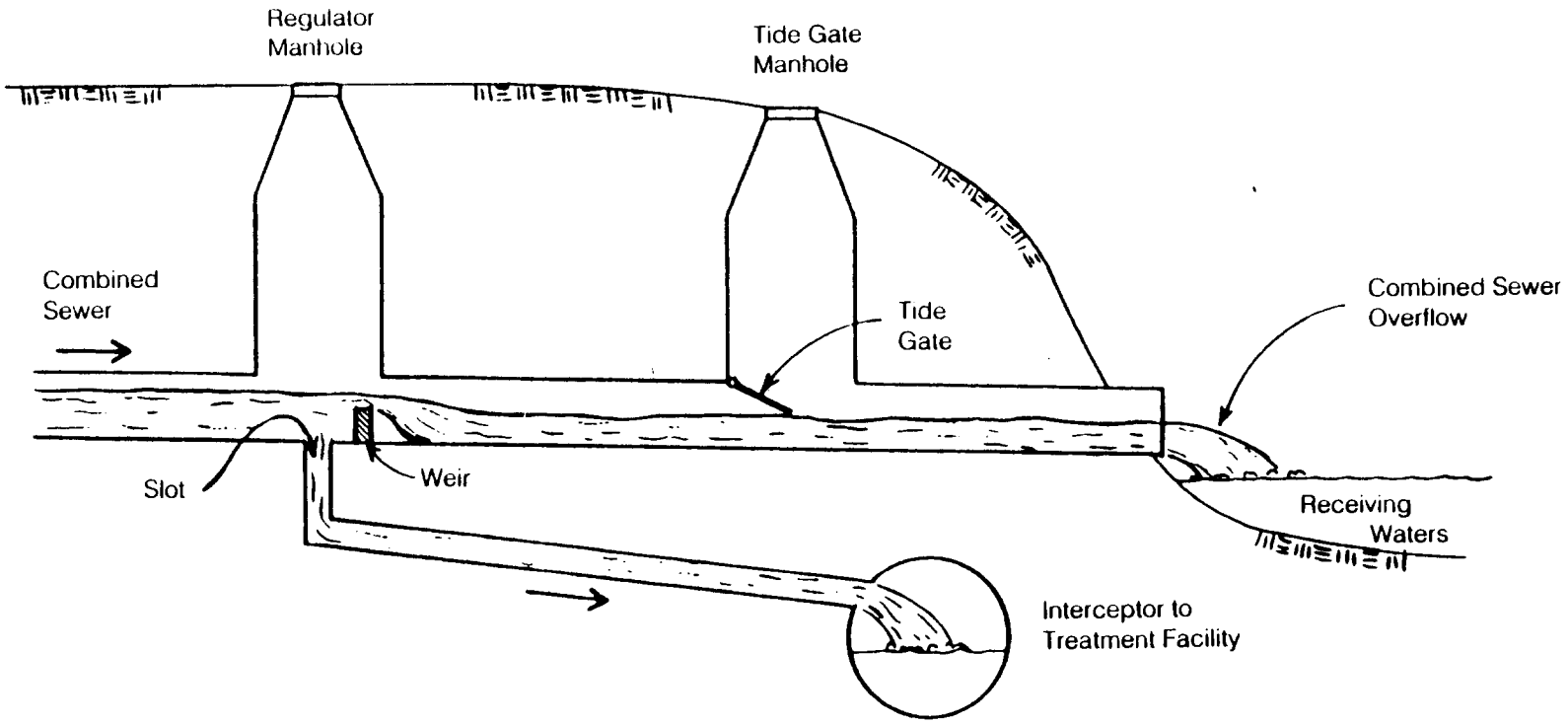

Figure 10.2—Fixed orifice flow regulator.

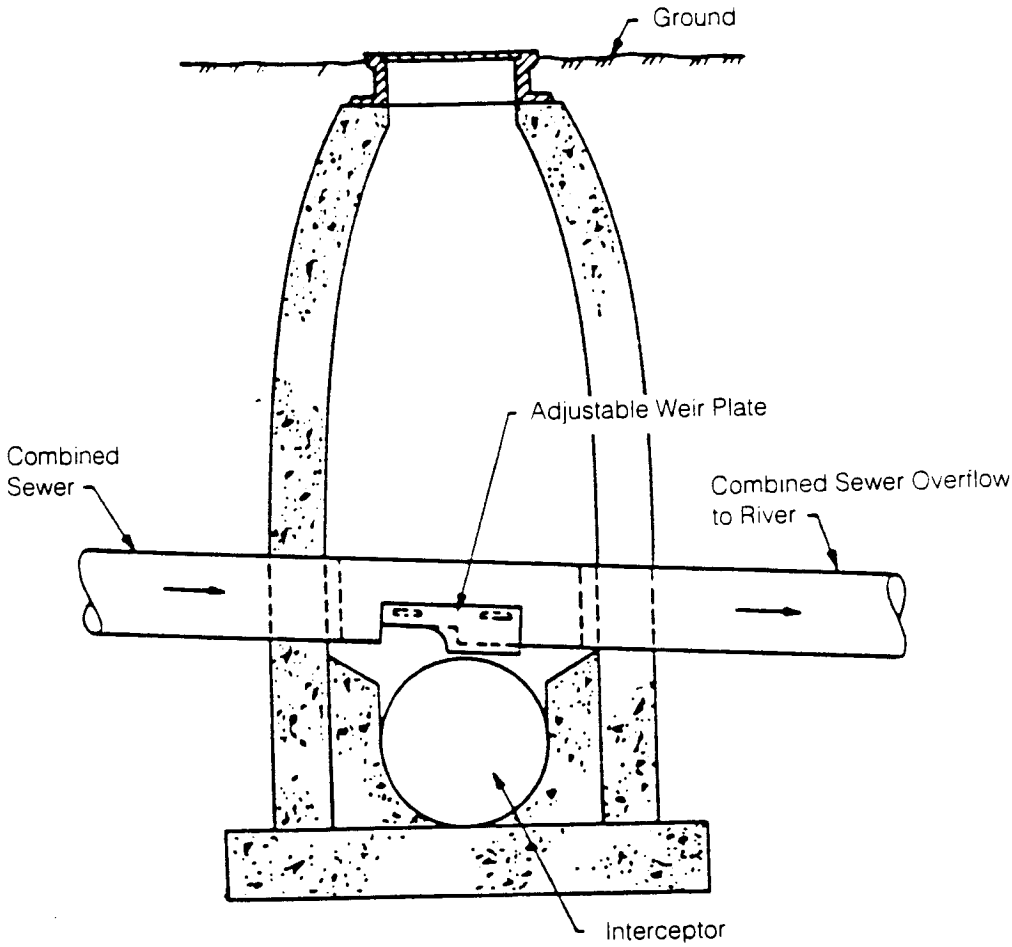

Figure 10.3—Leaping weir flow regulator (Metcalf & Eddy, 1981).

Manually Operated Gate This regulator has an orifice with a gate as shown on Figure 10.4. The orifice opening can be varied by manual adjustment of the gate. The regulator consists of two chambers: diversion chamber and orifice chamber. The diversion chamber has a low dam placed across the combined sewer to divert the flow to the orifice chamber. The orifice chamber has one or more orifices with shear or slide gates, either circular or square in shape (a square shape is less subject to clogging than a circular shape). The setting of the gate is based on the maximum allowable flow that can be diverted to the interceptor during a storm.

Side-Spill Weir This consists of a weir constructed parallel to, or at a slight angle to, the axis of the combined sewer. The height of the weir crest is such that the peak dry-weather flow continues in the sewer to the interceptor. During wet weather conditions, the combined sewage overflows the weir to the outfall, or to CSO storage/treatment. These weirs cannot generally control flow accurately, but they are in common use because of their simplicity and low cost.

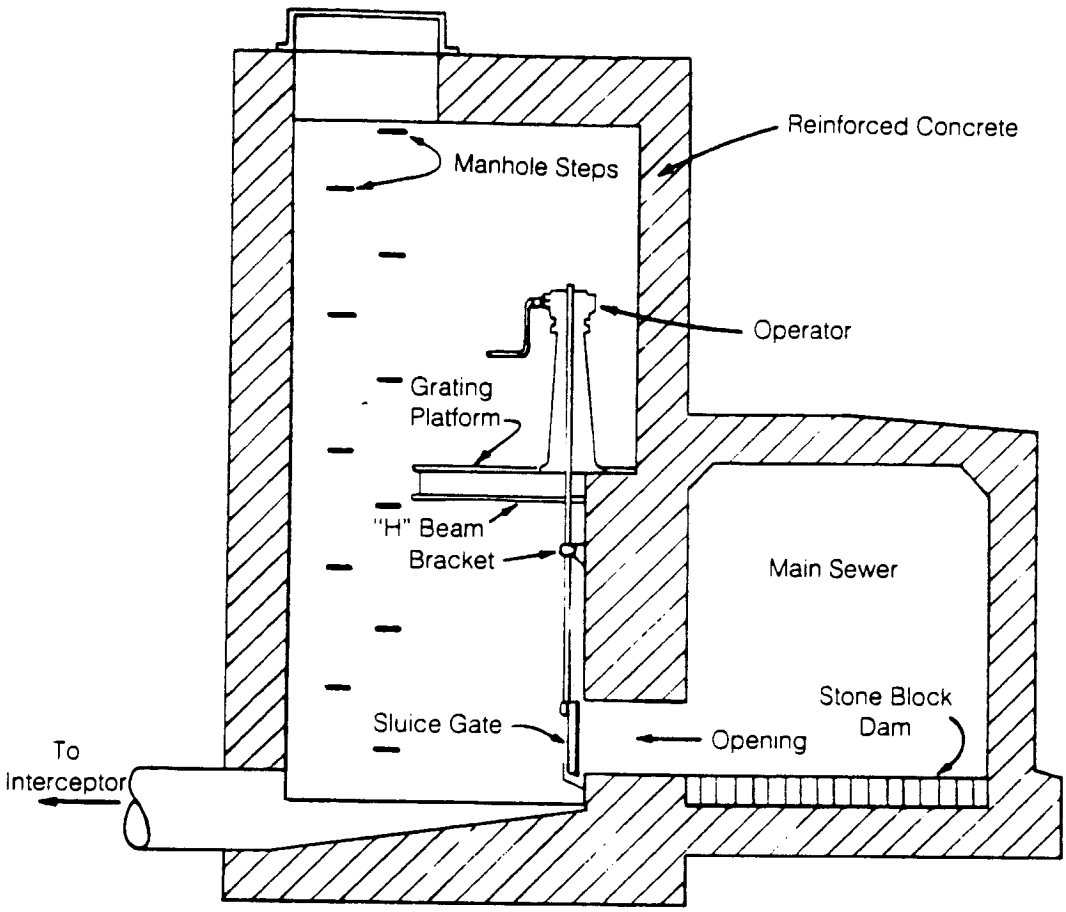

Figure 10.4—Manually operated gate regulator (APWA, 1970).

Vortex Regulator The vortex regulator is becoming widely used in combined sewer flow regulation. Its chief advantages are simplicity and the fact that it will produce a relatively constant discharge over a range of heads. See Water Pollution Control Federation (1989) for a detailed discussion.

3. *Dynamic Regulators*

There are two major types of dynamic regulators, semi-automatic and automatic.

Semi-automatic Dynamic Regulators Semi-automatic regulators do not require an external source of energy for operation. The flow conditions in the combined sewer, regulator, and/or interceptor actuate mechanisms to restrict the flow being intercepted. These types of regulators include:

(a) Float-Operated Gates: Automatic gates operated by floats that rise or fall as the water elevation in the sewer increases or decreases. When the interceptor is filled to its capacity, the gate could close entirely

cutting off the flow to the interceptor. In common application, the regulator restricts the flow from the combined sewer to the interceptor to a predetermined peak dry weather flow. The regulator, as shown on Figure 10.5, includes a diversion chamber, a regulating chamber, and a tide gate chamber, if required.

The diversion chamber has a low (preferably 6") weir perpendicular to the flow channel of the combined sewer for diverting dry-weather flow to the regulating chamber. Sometimes, the invert of the diversion chamber is depressed to prevent the overflow of peak dry-weather flow to the outfall. The regulating chamber houses the float, the regulating gate, and the connecting linkage between the float and the gate. The gate is installed on an opening in the common wall between the diversion and regulating chambers. The float is located in a well connected to the combined sewer or interceptor. For restricting the flow to the interceptor, the actuating mechanism operated by the float in the wet well connected to the combined sewer adjusts the gate opening to maintain the predetermined flow rate. In other types of arrangements

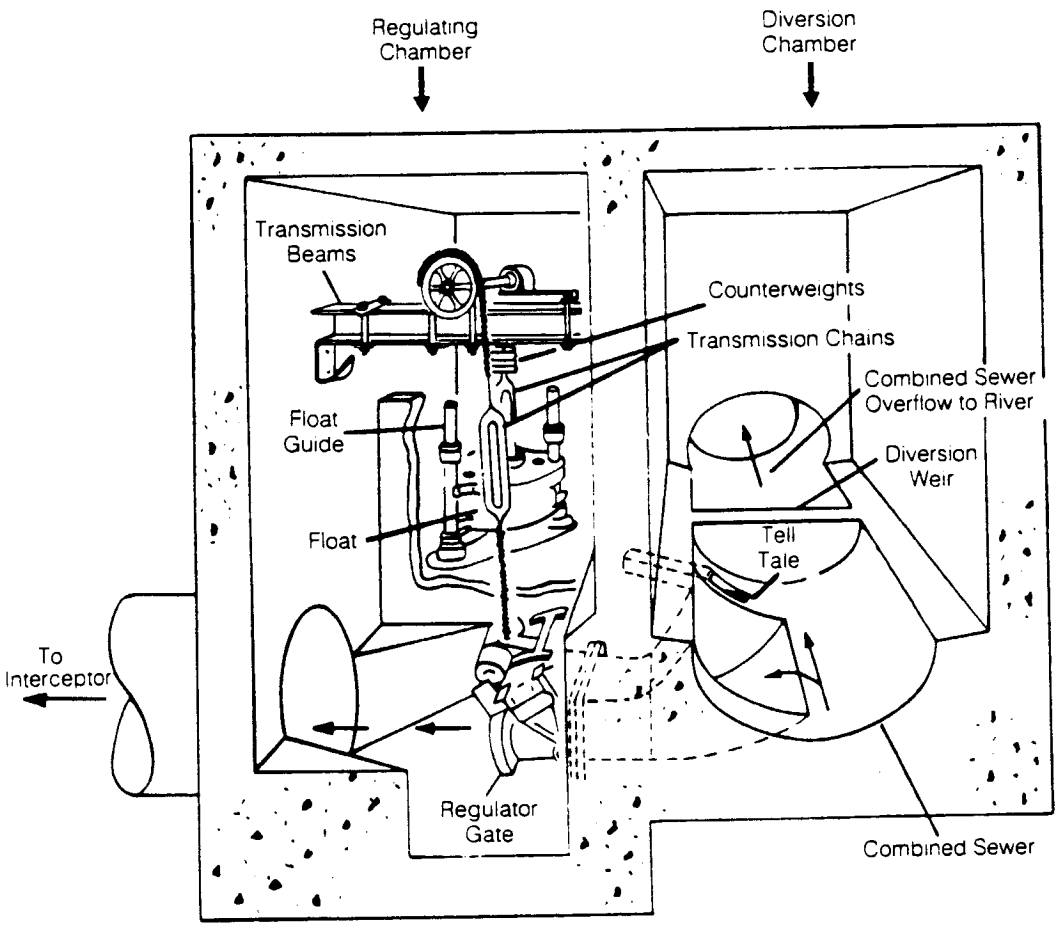

Figure 10.5—Typical float-operated mechanical regulator (Metcalf & Eddy, 1981).

where the CSOs are to be restricted, the float well is connected to the interceptor. The overflows do not occur until the interceptor is fully loaded to a predetermined level.

(b) Tipping-Gate Regulator: This regulator consists of a plate which is pivoted off-center as shown on Figure 10.6. The motion of the plate is controlled by the difference between upstream and downstream water levels. The regulator structure is similar to the one required for manually-operated gates. The tipping plate is mounted on the vertical orifice through which the flow is to be controlled. The regulator can be adjusted in the field by adjusting the gate stop disk which controls the gate opening. This regulator is generally used for flows greater than 4 cfs.

Automatic Dynamic Regulators Automatic dynamic regulators operate control gates using an external source of energy such as electricity or hydraulic power.

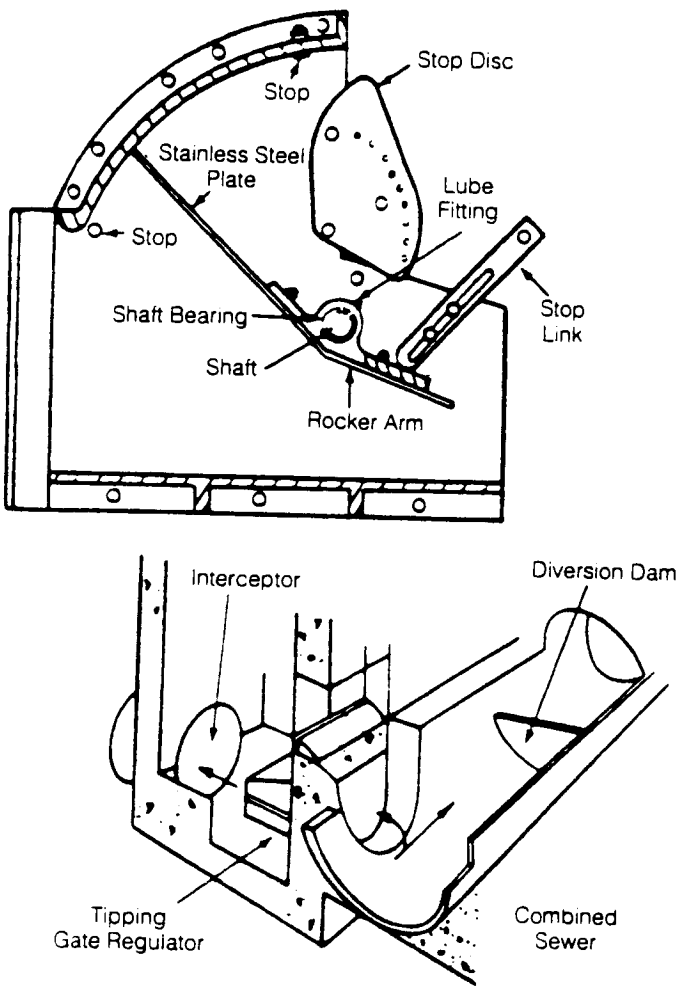

Figure 10.6—Tipping gate regulator (WPCF, 1989).

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(a) Motor-Operated Gates: This regulator consists of a low (typically 6") weir at the invert across the combined sewer which diverts the combined sewage to a vertical opening such as an orifice. The size of the opening is based upon the maximum flow to be diverted. Either a tainter gate, sluice gate, or shear gate is mounted on the orifice to control the flow. The gate is operated by an electric motor activated by a sensing probe. The probe records the sewage level in the control section and transmits the signal to start the motor when a predetermined level is reached in the control section.

The regulator has two to four chambers: diversion chamber, regulator chamber containing the gate and the sensor, a motor chamber, and a tide gate chamber where required. A typical motor-operated gate flow regulator is shown on Figure 10.7.

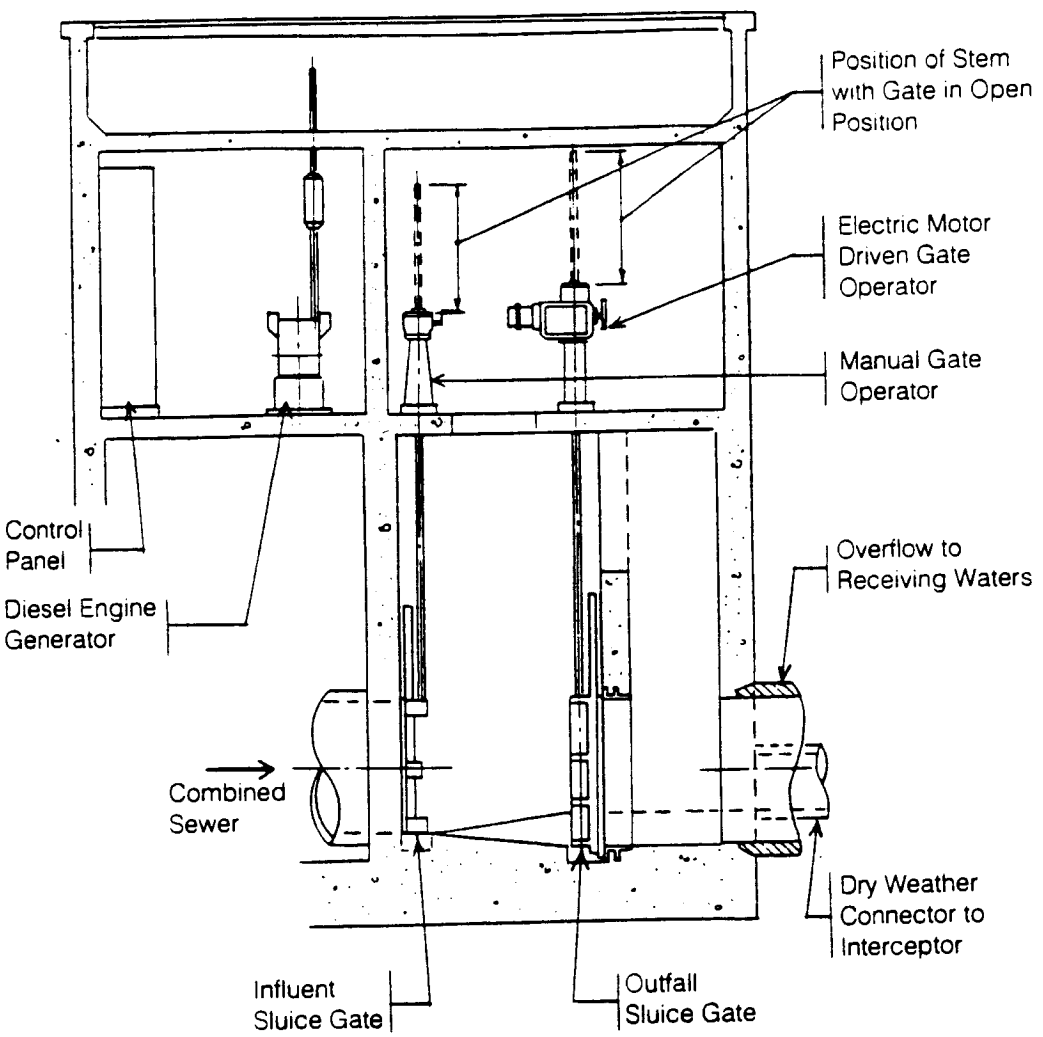

Figure 10.7—Typical motor-operated gate flow regulator (APWA, 1970).

Motor-operated regulators are used on larger combined sewer systems. They permit positive control of flow diverted to the interceptors as well as to outfalls. Remote control allows the storage within the sewer system to be used to minimize overflows as well as to prevent overloading of the treatment plant. This type of regulator is an integral part of many CSO abatement plans.

(b) Cylinder-Operated Gate Regulators: Similar to the motor-operated gate regulator, this gate is operated by hydraulic cylinder (which uses either oil, air or water). As the unit is self-enclosed, it can operate even when submerged.

C. Overflows

The purpose of an overflow structure is to convey combined sewage that exceeds the capacity of the intercepting sewer, CSO storage/treatment facilities and the wastewater treatment plant to receiving waters. Combined sewer outlets are usually located above the normal water surface of the receiving waters. However, in some cases, system hydraulic limitations require outlets to be submerged to a considerable depth.

1. Gravity Outfalls

Gravity outfalls are usually above the receiving water surface and are normally provided with a concrete or brick head wall. Riprap is typically provided to reduce erosion in the vicinity of the head wall. In some cases, the head walls are constructed on steel sheet piling.

Where receiving waters are subject to fluctuating water levels which result in submergence of the outlet, backwater (tide) gates are provided. These gates help to minimize the entry of receiving waters into the intercepting sewers and storage/treatment facilities during dry weather. During wet weather, the higher hydraulic grade in the sewerage system allows excess combined sewage to overflow into the receiving waters.

2. Submerged Outfalls

These outlets are partially or totally below the normal water surface of the receiving waters. Tide gates are installed to protect the intercepting sewers against flooding by receiving waters. In some cases, pumping stations are installed at the outfalls to provide positive drainage of combined sewers.

D. Interceptors

Intercepting sewers are designed to divert and convey dry-weather flow from combined sewers to the treatment plant. Regulators are used to divert and control flow into interceptors as discussed earlier.

The intercepting sewer typically is designed to convey the peak dry-weather flow, groundwater infiltration, and inflow generated within

the service area at ultimate development. As the peak dry-weather flow is typically 2–4 times the average dry weather flow for the design year, the intercepting sewers are designed for some larger multiple of the average dry-weather flow to intercept a portion of storm runoff. Interceptors are, in some cases, designed for up to 10 times the average dry-weather flow to assure capture of the "first flush." The first flush of stormwater contains solids which have settled in sewers during the antecedent dry periods in addition to the solids contained in the runoff itself. The interception of the first flush can be important for minimizing pollution of the receiving waters from CSOs. The first flush phenomenon can be as variable as CSOs themselves, and may be difficult to observe at the outlet of very large catchments.

II. COMBINED SEWER OVERFLOW POLLUTION LOADS

The impact of pollution from CSOs can be significant, since they are generally located in some of our most heavily populated urban centers. The quality of combined sewage varies dramatically based on a number of natural and man-made influences. Among the natural influences are rainfall characteristics and inter-event times, ground topography, and vegetation. Man-made influences include land use (residential, commercial, industrial), street sweeping practices, air quality, and type of street surface. A summary of observed pollutant concentrations in CSOs is presented in Table 10.1.

CSOs occur as a result of rainfall events. While the runoff dilutes the sanitary flow, it can have a considerable pollutant load of its own (see Chapter 5 for a discussion of runoff quality). In addition, storm flows cause a re-suspension of materials settled in sewers, which contributes to the so-called "first flush" phenomenon mentioned in the preceding section. For an excellent discussion of CSO pollution, see also the Water Pollution Control Federation Manual of Practice (1989).

In early studies of CSO pollution loads, one of the problems was that samples only were taken for the first few minutes of a storm. Consequently, the measured loads of suspended solids and BOD were very high. As more data over entire storm events became available, it was evident that the concentration of pollutants often decreased during the runoff event. When such conditions exist, it is important to relate concentrations to their corresponding runoff discharges when evaluating the impact on receiving waters. Since the late 1960s, advancements have been made in the state-of-the-art of predicting the quality of combined sewage, primarily in the field of mathematical modeling (see also Chapter 7).

Three levels of refinement may be employed when estimating the pollution load from a combined sewer area. These levels represent the transition from a relatively simple average yearly loading to a quite detailed representation of stormwater pollution during storm events.

TABLE 10.1. Observed Pollutant Concentrations in Combined Sewer Overflows.

	Average Pollutant Concentration, mg/L									
	TSS	VSS	BOD	COD	Kjeldahl Nitrogen	Total Nitrogen	PO_4-P	OPO_4-P	Lead	Fecal Coliforms[a]
Des Moines, Iowa	413	117	64	—	—	4.3	1.86	1.31	—	—
Milwaukee, Wisconsin	321	109	59	264	4.9	6.3	1.23	0.86	—	—
New York City, New York										
Newtown Creek	306	182	222	481	—	—	—	—	0.60	—
Spring Creek	347	—	111	358	—	16.6	4.5[b]	—	—	—
Poissy, France[c]	751	387	279	1005	—	43	17[b]	—	—	—
Racine, Wisconsin	551	154	158	—	—	—	2.78	0.92	—	201
Rochester, New York	273	—	75	—	2.6	—	—	0.88	0.14	1140
Average (not weighted)	370	140	115	367	3.8	9.1	1.95	1.00	0.37	670
Range	273–551	109–182	59–222	264–481	2.6–4.9	4.3–16.6	1.23–2.78	0.86–1.31	0.14–0.60	201–1140

[a]1000 organisms/1000 mL.
[b]Total P (not included in average).
[c]Not included in average because of high strength of municipal sewage when compared to the United States.
(Lager et al 1977).

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The three levels of refinement have been defined (EPA 1976) as listed below:

(a) Level 1—Average Annual Storm Load: This level of refinement estimates the total stormwater pollution load from an urban area on an average annual basis. It is generally most useful in assessing potential long-term water quality problems such as sediment deposition, nutrient loading, or chronic toxic effects. It also may be useful in obtaining an order of magnitude comparison between annual stormwater pollution loads and point source pollutant loads.
(b) Level 2—Storm Event Loads: This level of refinement estimates the actual distribution of storm event pollution loads throughout the year and indicates the variability of the total stormwater pollution load generated by each storm event. This level of detail may be required for an assessment of transient water quality problems such as dissolved oxygen or bacterial concentrations.
(c) Level 3—Load Variation Within Storm Events: This level of refinement describes stormwater pollution loads as a function of time within each storm event. This level represents the effect of storm patterns and intensities on stormwater pollution and will indicate whether there is a first flush of pollutants.

The selection of an appropriate level of refinement for defining stormwater pollution loads should be based on the level of detail required to assess receiving water impacts. The impacts of the various types of pollutants discharged to receiving waters have characteristic time and space scales (EPA 1976). In general, long-term water quality problems, such as sediment deposition or nutrient loading, can adequately be evaluated using average annual storm loads (level 1), whereas more reactive pollutants (e.g., coliform bacteria or oxygen-consuming materials) usually require storm event load determinations (level 2). A level 3 determination of the load variation within storm events may be required to properly design stormwater pollution control structures.

Wu and Ahlert (1978a and b) have proposed categorizing the numerous methods for estimating stormwater pollutant loads as:

a) Zero-order methods—An estimate of an area's stormwater pollutant loads based exclusively on data reported in the literature is termed "zero-order," which indicates the relative flexibility of estimation from reported data. The method is the most crude, can be inaccurate, and is generally only useful for determining level 1 average annual loads. The best source is data from the Nationwide Urban Runoff Program (NURP) of the U.S. EPA (EPA, 1983). This data is available from the USGS and the Illinois State Water Survey. Other sources of stormwater data include Huber et al. (1980), Manning et al. (1977), Sartor and Boyd (1972), Bradford (1979), Polls and Lanyon (1980), and Collins and Ridgway (1980).
b) Direct or rational methods—If storm runoff and pollutant concentration are independent, a direct or rational calculation of the stormwater loading rate is the product of a mean concentration and a mean discharge.

The mean discharge is calculated, and the mean concentration is estimated using one of several possible approaches, which include: (1) published data for a watershed with similar characteristics, i.e., zero-order, (2) observed field data from the watershed of concern, and (3) a regression equation, such as those proposed by Driver and Tasker (1988). The direct calculation of stormwater pollutant loads can be applied to estimate both level 1, average annual loads, and level 2, storm event loads. Documentation regarding use of the direct calculation method is presented by Driscoll et al. (1979), Young et al. (1978), and the U.S. Environmental Protection Agency (1976).

c) Statistical methods: Statistical methods employ techniques such as regression, correlation, and frequency analysis to predict stormwater pollutant loads. Site-specific factors normally considered in statistical methods include precipitation, watershed characteristics, land use patterns, and population densities. The best description can be found in the report on the NURP published by the U.S. Environmental Protection Agency (EPA 1983). Statistical methods can also be useful to examine field data from a site for the purpose of identifying parameters which affect nonpoint pollution loads (Griffin et al. 1978).

d) Descriptive or modeling methods: Descriptive methods for estimating stormwater pollution loads deal with deterministic rainfall/runoff mechanisms that affect the quality of a stormwater discharged from a watershed. These mechanisms can usually be represented by mathematical expressions which can be incorporated into programs for simulating stormwater quantity and quality on a digital computer.

III. METHODOLOGY FOR EVALUATING COMBINED SEWER OVERFLOWS

A. General

CSOs are evaluated generally on the basis of impacts to receiving water quality. Because of the complex and variable hydrologic, hydraulic, and water quality relationships associated with CSOs, mathematical simulation models are extensively used to facilitate these evaluations. As discussed in Chapter 7, these models simulate the physical characteristics of the combined sewerage system and, if properly calibrated, can be used to predict CSO volumes and pollutant loadings to a receiving water for a single storm event or over a long period. Receiving water models are used to predict pollutant levels in rivers, estuaries or bays resulting from CSO discharges. They can interface directly with models of runoff processes and simulate the effects of advection, dispersion and individual constituent changes such as decay or growth.

To assure accurate and representative models, input data must be complete and correct. Data base development includes the collection of all available information on the sewerage system, its tributary area and the receiving water(s). This information is validated and expanded, as required, through field investigations and surveys. All data is then

assembled and properly formatted for input into the various models. Computer data base management or spreadsheet programs are very useful (some are commercially available) for this purpose. In addition, schematic representations of the physical facilities within the system and, in some cases, the receiving waters are developed to graphically depict the models to facilitate their use.

Once developed, the land-based and receiving water models are calibrated and validated (see Chapter 7 for a description of this process). The land-based models can then be used with reasonable confidence to test system capacity under future dry-weather flow conditions, and to determine design flow rates for sizing CSO abatement facilities. Receiving water models can be used with somewhat less confidence to predict impacts of CSOs upon receiving water quality over a long-term period or for a specific storm event. The models together can also be used to test alternative CSO mitigation techniques (relative impacts).

B. Data Collection

Data collection is a formidable but very necessary and important task. Accurate and up-to-date information on the sewerage system and receiving waters is vital to assure proper model representation. Data collection procedures were described in Chapter 3, and the Water Pollution Control Federation Manual of Practice on Combined Sewer Overflow Pollution Abatement (WPCF 1989) provides excellent guidance.

C. Field Investigations

1. *Inspections of Physical Facilities*

Field inspections serve to verify and expand on information obtained under the data collection task. Typically, this includes obtaining or verifying dimensional data and determining the physical condition and operating deficiencies of regulators, tide gates, pumping stations, overflow pipes and other major system facilities. Internal television inspection of major interceptors is suggested to determine structural condition, location of major sources of infiltration and the extent of solids (grit) deposition. This latter item is vital since significant grit deposition reduces interceptor carrying capacity. This must be accounted for when calibrating the land-based computer models to assure accurate system representation.

2. *Flow Monitoring and Sampling*

Data on sewer system flow rates and wastewater quality under both dry weather and wet weather conditions is required for model calibration and to obtain a thorough understanding of system operation and response to wet-weather events. The selection of flow monitoring and sampling sites is dependent upon several factors, including the number

of drainage basins within the system, land use within each basin, unmetered system boundary points (i.e., locations where wastewater enters the study area systems from adjacent, upstream systems and/or locations where wastewater leaves the study area system) and the extent of in-place flow and quality measurement facilities at boundary points, pumping stations, or wastewater treatment plants.

For systems having a large number of drainage basins and CSOs, it is not economically feasible nor is it necessary to monitor and sample each basin. Typically, a manageable number (3–5) are selected that represent a cross-section of land use throughout the entire system. Under dry-weather conditions, wastewater flow from each selected basin is continuously measured for at least a one-week period to determine average, maximum and minimum flow rates and to estimate infiltration. Samples are also taken over at least a 24-hour period and typically analyzed for conventional parameters and, if warranted, other parameters that may be of concern to a particular receiving water (metals, PCBs, nutrients, etc.). This information, coupled with flow estimates, can be used to estimate dry-weather flow and quality in other, nonmeasured drainage basins. During this monitoring and sampling period, continuous flow gaging on major interceptors is warranted to verify total system dry weather flow estimates, and to achieve a system-wide flow balance. All dry-weather flow and quality data serve as base-flow input data to the land-based models.

A combined sewer system wet weather monitoring and sampling program is also conducted to determine the system's hydraulic response to a specific storm event, and the rate and quality of pollutant washoff from specific drainage basins. During several storm events (usually 3–5), continuous flow gauging is conducted at regulators or on overflow pipes to measure the rate and determine volumes of CSOs generated from the selected study area drainage basins. Samples are also taken at each CSO gauging location at specific time intervals following the start of the overflow. Sampling intervals are more frequent during the early stages of the overflow to determine whether there are water quality impacts due to "first flush" effects. In most cases, samples are analyzed for the same parameters measured for the dry-weather sampling program.

To assure that CSO monitoring and sampling results are typical and representative, criteria for selecting wet-weather events for conducting this field program must be established. Typically, the representativeness of each storm is based on antecedent dry period and the length and intensity of precipitation. Events occurring after a dry period of at least three days, that last a minimum of four hours and have intensities that produce measurable runoff are usually considered representative. A rain gauge located within the study area (if available) is used to develop storm hyetographs.

Information obtained from the wet-weather monitoring and sampling program is used to calibrate the land-based models.

3. *Receiving Water Sampling*

Receiving water sampling typically coincides with land-based monitoring and sampling under dry-weather conditions and during wet-weather events. For rivers and streams, flow monitoring may also be required. However, in many cases, stream flow data from existing gauges installed and maintained by the USGS or others can be used (and flow data adjusted as required) if located a reasonable distance from the study area.

Receiving water quality data is often available from a state authority, local planning agency or local university. If sampling is required, several aspects should be considered. Sampling locations are generally dependent on the type of receiving water (river, bay, or estuary), location of CSO discharges, model calibration needs and site-specific concerns and issues such as shellfish bed or beach closure problems. For rivers, it is important to measure quality at boundary points and at points upstream and downstream of the major CSO outlet(s). In bays or estuaries, sampling locations are more dependent on modeling needs. Some bay models do not generate results as precise as required near shorelines where, in many instances, CSO pollution is greatest. Consequently, more intensive near-shore sampling is sometimes warranted to assist in evaluating CSO impacts.

Pollutants of concern and, in turn, laboratory analytical requirements, are specific to the study area and receiving water issues and concerns. Common parameters include bacteria, solids, dissolved oxygen, BOD, pH, and selected metals.

D. Assessment of Existing and Future Land-Based Conditions

1. *Population and Dry-weather Flows*

Information on dry-weather flows in a wastewater collection system is essential to estimations of the quantity and quality of CSOs. Although the quantity of dry-weather flow is relatively minor in comparison to the large volume of stormwater required to produce an overflow, its high contaminant level has a significant impact on the quality of CSOs.

Dry-weather flow data is used in several ways during CSO evaluations. It is used with wet-weather data for calibration of the system computer models. Estimated average dry-weather flows are used to approximate available in-system storage capacity and simulate the mixing of dry-weather flow and stormwater runoff, providing estimates of overflow quality and quantity at each CSO outlet. Monitoring and sampling at the CSO outlets, discussed in the preceding section, provides verification of the mixed flows derived from the model.

An analysis of the existing and estimated future population and water consumption of the study area is necessary to estimate the area's dry-weather wastewater flow rates. Per capita water consumption estimates, obtained from metered water consumption data, multiplied by esti-

of drainage basins within the system, land use within each basin, unmetered system boundary points (i.e., locations where wastewater enters the study area systems from adjacent, upstream systems and/or locations where wastewater leaves the study area system) and the extent of in-place flow and quality measurement facilities at boundary points, pumping stations, or wastewater treatment plants.

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mated population, results in an estimate of total average daily water consumption on a drainage basin or study area basis. This information is used to estimate average daily domestic wastewater flows, and when added to estimates of commercial, industrial, and institutional flows and infiltration, results in the estimated total average dry-weather flow. Dry-weather flow estimates should be confirmed with field measurements, since calculated dry-weather flows are based on several assumptions and estimates associated with population, water consumption, and infiltration.

2. *Land Use and Zoning*

Land use is directly related to the quantity of runoff expected to flow to receiving waters during wet-weather events. The type and quantity of pollutants contained in runoff varies considerably from one land use category to another, and a thorough knowledge of existing and potential future land uses is an important element of a CSO evaluation.

Depending on population densities, residential areas may contribute high coliform concentrations to CSOs due to the relatively high volume of domestic sanitary sewage discharged into the combined sewer system. Industrial areas may pose additional problems from toxic constituents, including the discharge of heavy metals, volatile organics, polychlorinated biphenyls (PCBs), and other toxic substances. Industries may also add to the total volume of combined sewer overflow if they contribute a major portion of the dry weather flow.

Parks and other unpaved open areas serve to reduce the amount of runoff that drains to either a combined sewer or storm drain because much of the rainwater that falls on grass-covered areas is able to infiltrate into the ground. However, open areas or parks that are not properly graded, and/or do not have sufficient grass cover, can increase the grit and suspended solids content of the runoff as a result of soil erosion. This can lead to sediment deposits within conduits and a high solids content in CSOs and their receiving water.

Areas designated for institutional land use can increase the pollutant content of CSOs in several ways. Institutional areas containing schools, churches, hospitals, or municipal buildings have a potential for discharging large slugs of sewage and associated pollutants to the collection system. If these discharges occur during a rainfall event, the sanitary portion of the total flow in the combined sewer can increase, increasing the pollutant concentrations of CSOs where they occur. Hospitals have the additional potential of producing waste streams containing high concentrations of pathogenic organisms that may become a health problem when discharged in the vicinity of bathing or shellfish areas.

Runoff from highways, major roadways and railroads must also be addressed. Oil, grease, and other automotive pollutants are usually present on road surfaces in sufficient quantities to contaminate storm-

water runoff that eventually discharges to receiving waters. Hydrocarbons from engine exhaust systems plus sanding and salting operations during the winter season can also add contaminants to roadway runoff.

E. Definition of Receiving Water Issues and Goals

The techniques used to model receiving waters must be designed to address the local issues and federally approved water quality standards. "Issues" refer to the questions and concerns from the public and regulatory agencies regarding CSO pollution.

CSO abatement goals should be established to meet designated uses of receiving waters for all but an acceptable number of relatively infrequent events. Receiving waters can be designated for uses such as public water supply, recreation, protection and propagation of aquatic life, agriculture, and navigation. CSOs should not discharge directly into a public water supply, but generally can discharge to waters with other uses after suitable controls are in place to minimize the frequency, volume, and pollutant concentration of the discharge. The acceptable degree of control is usually established by a local, regional, or state agency with jurisdiction to protect water quality.

Wet weather water quality conditions are best evaluated in a probabilistic context. To do this, modeling must be done over some reasonable period of record. The use of specific historical records is obviously ideal, but synthetic rainfall/runoff sequence generation may be required if real data is unavailable or sparse. The purpose of such analyses is to quantify the relationship between runoff characteristics (intensity, duration, time of year), and the risk (probability) of receiving water quality problems. These analyses provide the framework for practical decision making and risk analysis (see also Chapter 8).

IV. COMBINED SEWER OVERFLOW MITIGATION TECHNIQUES

A. General

Techniques for reducing CSO pollution can be grouped into three broad categories; 1) source controls to reduce the quantity of pollutants entering the system, 2) collection system controls designed to increase the system's effectiveness in conveying and/or storing excess flow, and 3) off-line storage and treatment to remove pollutants from the overflow. The technique selected for any given situation may include components from each of these categories. Although not considered a separate improvement category, system maintenance is an essential part of any pollution control program.

This section discusses the identification of applicable control techniques, maintenance implications, and describes the different types of facilities and how to evaluate their effectiveness. For an extensive dis-

cussion of these issues, see the Water Pollution Control federation Manual of Practice (1989).

B. Identification of Applicable CSO Mitigation Techniques

Selection of CSO mitigation techniques is a very complex process. Not only are there numerous possibilities within each of the three major classifications noted above, but regulatory agency guidance, water use classification, and funding availability all must be considered. Further, some beneficial uses of receiving waters can be more economically addressed by treatment plant improvements.

1. Source Controls

Most source controls are nonstructural in nature, and are sometimes referred to as Best Management Practices (BMPs). Their common denominator is reduction of pollutant load which can reach the combined sewers and/or regulators. Control of illicit connections, street sweeping, catchbasin cleaning, waste oil collection, sewer cleaning/flushing, and control of runoff from oil storage facilities, car wash establishments, body shops, and the like, are examples. Another type of source control involves land use planning, which can impose limits on the flow volume and/or rate which can be conveyed to the combined sewer.

2. Collection System Controls

Generally, collection system controls are intended to ensure that the sewers are operating at peak efficiency. Elimination of illicit connections, adequate maintenance, and a complete understanding of how the system functions are nonstructural actions which should be the first step in a CSO control program. There then follows a set of possible improvements including regulator consolidation (conveying flow from several small structures to one large regulator) and control, storage, and/or treatment, both on- and off-line, and sewer separation (though from a pollution control standpoint, this generally is not an effective solution).

3. Off-line Storage and Treatment

These types of CSO abatement facilities are structurally intensive and costly, and are used as necessary following implementation of source controls and system optimization measures. Off-line storage can be provided using either underground or above-ground tanks. Because the runoff to be accommodated can vary greatly, it is most efficient to subdivide the tanks and to provide gravity flow into the unit. Treatment can vary from simple screening to complete treatment at a wastewater treatment plant.

C. Combined Sewer System Maintenance

In addition to normal maintenance activities involving sewers, regulators, manholes, pumping stations, metering facilities, etc., combined sewer systems have special maintenance needs involving regulators and tide gates. If CSO mitigation facilities have been installed, there is a substantially greater maintenance effort required. In particular, regulators can be difficult to maintain because of the harsh environment in which they must operate.

1. Records

Experience indicates that sewer maintenance records (or even records of regulator locations) are generally poor or nonexistent. Part of the problem can be traced to the age of the system, because most combined sewers are 40 or more years old. In the interim, field modifications and other changes may have resulted in a system that often bears little resemblance to that shown on original record drawings—if the drawings still exist.

Before the system can be maintained, it must be defined. Therefore, the first step in a maintenance program must be the development of a record system that not only describes what currently exists but also is easily updated to reflect new information that becomes available, and/or changes made by field personnel. As noted earlier, various computer programs are available which greatly aid in this process.

2. Regulators

By their very nature, regulators are prone to failure. The incoming combined sewer is larger than the dry-weather outlet and, as a result, the regulator is subject to clogging. All regulators should be inspected on a periodic basis. Those that are a particular problem, as demonstrated by maintenance and/or complaint records, must be inspected more often.

D. Nonstructural Techniques

Nonstructural CSO mitigation techniques focus on the operation of the existing combined system or regulatory control to minimize overflow quantity and pollution load. Nonstructural techniques can be an important element of a CSO mitigation plan, though significant reductions in CSO pollution for highly urbanized areas may not be achievable solely through nonstructural control.

1. Land Use

Historically, land use controls focused at mitigating CSOs have typically not been used, since most combined sewer areas are already developed. However, rationally conceived long-range land use programs can reduce or prevent the aggravation of CSO problems. The

development or redevelopment of vacant land within a combined sewer area can provide a mechanism to regulate the stormwater runoff to a combined sewer system. Increasing open space and greenway areas (if possible) and/or the provision of detention facilities can both reduce the rate and quantity of stormwater runoff and decrease the nonpoint pollution load contribution to CSOs.

2. *Interceptor Sediment Removal*

Combined sewers, and the interceptors to which they are tributary, experience large ranges of flow velocity. During periods of low velocity (two ft/sec or less), sedimentation occurs. Additionally, irregularities in sewers such as cracks, breaks, misalignments, or root intrusions can reduce velocities and enhance solids deposition. While sediment removal by cleaning will increase the efficiency of the interceptor and somewhat reduce CSOs, the reduction will be short-term unless the factors causing low flow velocities and sedimentation are permanently corrected.

3. *Regulator Modifications*

Regulator modifications can be used to use the capacity of the interceptor system more efficiently and thereby mitigate combined sewer overflows. The proper maintenance and operation of regulators is one of the most important management practices that can be used for the reduction of CSOs. Regulators should be operated to assure that all dry-weather flow is intercepted and no dry-weather overflows occur.

4. *Interceptor Surcharging*

Interceptors are designed to convey all dry-weather flow from tributary combined sewers, and combined sewage and runoff in excess of the dry-weather flow during wet weather. The surcharging of an interceptor allows for flows in excess of design limits to be delivered to downstream outfalls, pumping stations, or the treatment plant. Interceptor surcharging can cause sewer backups and flooding if the hydraulic grade line is higher than the ground surface in low areas that are tributary to the system.

5. *Inflow Diversion*

Often downspouts from roof gutters are connected directly to combined sewer systems. Where adequate pervious area and drainage exists, downspout disconnection can reduce the stormwater inflow to the sewer system and reduce CSO volumes and pollution. Sump pumps for foundation and other clear water drains can also be directed to pervious areas.

E. Structural Alternatives

The growth of urban and combined sewer areas has often been underestimated in the design of combined sewers. Many times new areas have been annexed and served by main sewers and interceptors that had been designed for outdated and smaller system boundaries. Additionally, with development, the proportion of impervious area has increased with corresponding increase in stormwater runoff. As a consequence, most combined sewer systems do not have adequate conveyance and storage capacity, and the application of system maintenance and nonstructural techniques often is not adequate to allow attainment of applicable water quality standards. Generally, structural measures are required to address the inadequacies of large combined sewer systems and to solve major CSO problems.

Because of the great variations in combined sewer system characteristics, engineering studies are required to select appropriate CSO mitigation solutions. For large systems, water quality, hydraulic, and hydrologic modeling is often necessary as discussed earlier (see also WPCF 1989).

1. Storage

Storage involves the containment of combined sewage that normally would discharge to receiving waters (see also Chapter 11 for a broader general discussion of storage). When excess flow capacity is again available in the interceptor system, the stored combined sewage is discharged into the interceptor and conveyed to a treatment facility. Storage facilities have been used extensively for CSO mitigation, particularly in Europe (Lager et al. 1977; Brombach 1990). Three types of storage are:

Detention/Retention Detention facilities are used to regulate or prevent stormwater runoff from entering a combined sewer system. By controlling peak stormwater flows, CSOs can be decreased or eliminated. As relatively unpolluted stormwater runoff is captured and stored, detention facilities can be sited in most urban settings, and are extensively used to attenuate peak runoff flows in separated sewer areas.

Combined sewer system detention facility design considerations include anticipated precipitation (including consideration of storms of long duration with severe antecedent conditions), runoff rates, sewer system capacities, treatment plant capacities and CSO control standards. Additionally, safety, operation, maintenance, aesthetics, and mosquito control must be addressed. Care must be exercised to insure against the sewage contamination of the detention facility as the requirements for off-line combined sewage storage are not the same as those for detention. Detention facilities are often multi-purpose.

Ponding is the most frequently used detention practice. Typically, the release of water from a pond to the combined sewer system is controlled by a small diameter pipe, restrictor, or orifice, and the release

of water is by gravity. When runoff from the pond's tributary area exceeds the release rate of the pond, water is impounded. Orifice sizes should be adequate to prevent blockages. The design release rate is a function of the depth of water and the orifice or restrictor size, and the required storage capacity is a function of the size and topography of the tributary area, land use, sewer system capacities and the level of CSO control required. Detention facilities can also be de-watered by pumping.

Sedimentation occurs in detention ponds, though in urban combined sewer areas, the sediment load may not significantly impact pond storage capacities.

Inline Storage Inline storage uses the volume of a combined sewer and interceptor system that is not being used to transport combined sewage to accommodate the storage of additional stormwater runoff. Often the areal distribution of runoff results in CSOs, even though excess sewer capacity is present in portions of the system. A collector system which nominally has excess wet-weather system volume could most likely use inline storage.

Off-line Storage Off-line storage involves the capture and storage of CSOs in tanks (usually large underground concrete tanks or tunnels), with the stored volume released (or pumped) into the interceptor system when interceptor capacity is available. This type of CSO storage/treatment is intended to mitigate or eliminate CSOs generated by storms up to a specific intensity and/or duration, or to capture the first flush.

Most large combined sewer systems require off-line storage to eliminate or significantly reduce CSO pollution. Off-line storage facilities have demonstrated their effectiveness in controlling storm and CSOs. Many regional plans include storage or combinations of storage alternatives as an integral part of the overall control process.

Off-line storage systems can also provide for the following:

(a) The attenuation of combined sewage peak flows.
(b) More efficient use of existing treatment capacity.
(c) Improved treatment plant average effluent quality.
(d) The elimination or reduction of sewer backups.

Because off-line facilities store combined sewage, consideration must be given to preventing septicity or handling the stored wastewater in a septic state. Additionally, the deposition of solids in the storage facility requires designers to provide adequate means for solids removal.

Off-line storage can be used to reduce the required size of a treatment facility. Wastewater treatment plants are typically designed with excess capacity for diurnal, seasonal, wet weather or other characteristic flow variations. Plants with tributary combined systems have been required by the U.S. EPA National Pollution Discharge Elimination System (NPDES) permits to have 2.5 times average dry-weather treatment capacity. There is an optimum combination of storage and treatment for

the most cost-effective system. As treatment capital and operational costs increase, it becomes more favorable to increase the storage capacity.

The off-line storage of combined sewage will result in the sedimentation of solids unless the storage facility has provisions for mixing. Accumulated solids can be washed out of the storage facility by inducing scouring velocities or agitation during the evacuation of the captured wastewater or by flushing with a potable or effluent water supply.

Storage facilities can be designed to hold the settled solids with periodic cleaning by dredging or other means, though prolonged storage can result in the settled solids becoming anaerobic with resulting odor problems. It is preferable to provide facilities for flushing out solids after each runoff event. When the solids are held in storage, some primary treatment (sedimentation) may be provided. The reduction of suspended solids could warrant the by-passing of primary unit operation during treatment.

Long detention times will also significantly reduce the BOD concentration (though if allowed to go anaerobic, BOD could increase), as well as reduce the dissolved oxygen concentration of the captured combined sewage and increase ammonia concentrations. Typically, the combined sewage stream from the off-line storage facility is blended with dry-weather influent prior to preliminary treatment to mitigate the possible impacts on the treatment unit operations of a relatively weak strength influent. If sedimentation is provided by an off-line facility, the discharge flows can be directed to bypass preliminary and primary treatment, and be directed to the secondary treatment unit operations, or if the flow quality permits, the discharge can be directed to receiving waters.

While some off-line storage facility designs will allow some natural aeration, average combined sewage BOD concentrations will still exceed the initial dissolved oxygen concentrations. As a consequence, captured combined sewage must be artificially aerated at the storage facility or evacuated for treatment in a timely manner to prevent septicity. To prevent the formation of odorous, noxious, explosive, and toxic gases, off-line storage facilities must maintain the stored combined sewage in an aerobic or facultative state.

2. *CSO System Control*

The control of combined sewer system operational components such as regulators and pumps to optimize system operation (minimize overflow, maximize capacity, minimize flooding) has been practiced for some time (Leiser 1974; Schilling 1986). Control schemes typically involve three levels of hierarchy:

(a) Local Control—which involves closed-loop control of regulators, gates or pumps.
(b) Regional Control—which provides coordinated operation of several local controllers.

(c) Global Control—which provides coordination of regional controllers to optimize system operation. Global control systems typically have the following components:
   (1) A data gathering system for rainfall, pumping rates, treatment capacity and regulator settings.
   (2) A telemetered control system for the manipulation of regulators, gates and pumps.
   (3) A computer system, with related mathematical models, for the centralized processing of system data and system control.

Local control is quite common, such as the use of a float controller to turn pumps on and off. Real-time control, on a regional or global basis, involves the use of **currently-monitored** process data (such as rainfall, flow, status of storage, wet well elevations, etc.) to operate regulators to achieve better (or optimal) systems performance **during** the actual process. Recent advances in microcomputers, weather radar, and control devices, as well as a growing body of experience in developing and operating real-time systems (Doering et al. 1987), have demonstrated that such control can provide significant improvements in system operation, although it can be difficult and costly to implement.

3. *Treatment*

Generally, interceptor capacities are inadequate to convey high combined sewage flows to the treatment facility, and even if one could convey the flows, it would be difficult to adapt the delicate microbial population associated with secondary treatment to the relatively short duration, high flow rates during runoff events. Treatment at individual CSO structures or for several consolidated overflows may be considered in conjunction with other CSO mitigation systems, when NPDES permit and state standards can be attained, and where the resulting head losses are acceptable.

The local treatment of CSOs has been primarily directed at the removal of settleable and suspended solids and floatable material, and disinfection. Reductions in suspended matter of 36–65% and in BOD of 25–40% are reported as achievable with primary treatment (Merritt 1968). Table 10.2 gives a comparison of typical physical treatment removal efficiencies for selected pollutant parameters (Lager et al. 1977). Disinfection is another local treatment process. A local treatment facility can use a combination of treatment processes.

(a) Sedimentation—The major objective of sedimentation is to produce a clarified effluent by gravitational settling. It is one of the most common and well established wastewater treatment unit operations. Sedimentation tanks provide some storage capacity, and disinfection can be concurrently effected. Sedimentation facilities can be used in conjunction with the addition of chemicals for improved removal of solids by coagulation (Graham 1978).

TABLE 10.2. Comparison of Typical Physical Treatment Removal Efficiencies for Selected Pollutant Parameters.

Physical Unit Process	Percent Reduction					
	Suspended Solids	BOD_5	COD	Settleable Solids	Total Phosphorus	Total Kjeldahl Nitrogen
Sedimentation						
Without chemicals	20-60	30	34	30-90	20	38
Chemically assisted	68	68	45	.....	..	..
Swirl concentrator/ flow regulator	40-60	25-60	..	50-90	..	..
Screening						
Microscreens	50-95	10-50	35	.....	20	30
Drum screen	30-55	10-40	25	60	10	17
Rotary screens	20-35	1-30	15	70-95	12	10
Disc strainers	10-45	5-20	15	.....	..	..
Static screens	5-25	0-20	13	10-60	10	8
Dissolved air flotation[a]	45-85	30-80	55	93[b]	55	35
High rate filtration[a]	50-80	20-55	40	55-95	50	21
High gradient magnetic separation[d]	92-98	90-98	75	99	..	..

[a]Process efficiencies include both prescreening and dissolved air flotation with chemical addition.
[b]From pilot plant analysis.
[c]Includes chemical addition.
[d]From bench scale and small scale pilot plant operation, 1 to 4 L/min (0.26 to 1.06 gal/min).

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Sedimentation facilities have a relatively high land requirement, the captured primary sludge must be appropriately handled, and the solids removal efficiency is a function of the combined sewage influent flow rate, which can vary significantly.

(b) Swirl and helical concentrators—Swirl and helical concentrators regulate both the quantity and quality of combined sewage. Solids separation is caused by the inertia differential resulting from a nonlinear path of flow travel. The flow is separated into an overflow, and a concentrated low volume of wastewater that is intercepted for treatment at the treatment plant. Interceptor or storage capacity must be available for the concentrated effluent.

(c) Screening—Screening provides high rate separation of particulate matter from combined sewage. Pretreatment screening devices are most appropriately used for CSO applications. These include drum, rotary, and static screens. Removal efficiencies range from 5–20% of the suspended matter. Removal efficiency tends to increase as influent suspended solid concentrations increase. Only small removal increases are effected by the use of additional screens of the same size in series.

(d) Dissolved air floatation—Dissolved air floatation uses the small air bubbles that form on suspended particulate matter to float the particulate matter for removal. After the combined sewage is pressurized with air, depressurization releases the air as small bubbles. Oil, grease, and other floatables can also be removed. Small and light suspended matter can be removed more efficiently and quickly than by sedimentation. Chemical addition is sometimes used to improve removal efficiency. Operating costs are relatively high due to the compressed air requirement, and the process is sensitive to operational control.

(e) High rate filtration—High rate filtration has been used to capture suspended solids on a fixed bed for anthracite coal and sand. High rate filtration was developed for the treatment of industrial wastes. Backwashing is used to prevent the clogging of the filter. Pretreatment may be required to remove coarse solids.

(f) Disinfection—Disinfection does not quantitatively remove any of the organic or inorganic combined sewage pollution. Disinfection is used to control pathogenic organisms, and to prevent significant populations of living pathogenic organisms from entering receiving waters. It is often required as minimum treatment for CSOs. Generally, disinfection is effected by adding chemical oxidizers to the overflow. Chlorine and sodium hypochlorite are among the chemicals used. Adequate mixing and detention time must be provided, and dechlorination may be required.

4. *Sewer Separation*

Sewer separation can be defined as the division of an existing combined sewer system into non-interconnected sanitary and storm sewer systems. The sanitary sewer system is tributary to a wastewater treatment facility, and the storm sewer system discharges directly to receiving waters. Complete sewer separation will eliminate CSOs. However, heavily urbanized areas can generate significantly polluted

stormwater runoff, and the negative impacts on a receiving stream may be significant. In fact, some European countries which have made major investments in sewer separation over the past decade have now decided that this practice is not environmentally acceptable. These countries have prohibited further separation, concentrating instead on source control and treatment of combined sewage.

Generally, sewer separation is effected by maintaining the existing combined sewers as either the storm or sanitary component of the new separated system. CSO mitigation may also be achieved by partial separation. The construction of new storm sewers to relieve selected portions of the combined system can reduce CSOs. Disconnection of rain leaders should also be considered if sewer separation is a viable alternative. Flow from rain leaders can be a significant portion of the stormwater entering a combined system.

F. Selection of CSO Mitigation Techniques

Mitigation techniques for CSOs include a number of possible technologies (discussed previously), as well as a range of degrees of control. Technologies include structural solutions—storage, treatment or conveyance facilities, and nonstructural or minimal structural solutions—source control programs, combined sewer operation and maintenance practices. Controls can be implemented to protect the receiving water quality for a design runoff return period, such as 1, 2, or 10 years. The objective of evaluations of CSO mitigation techniques is to determine the most practical and effective combination of technology and degree of control to achieve the water quality goals.

A cost-effectiveness analysis is usually performed as a means of collectively evaluating all of the factors of CSO control technologies, degree of control, and receiving water impacts. In such an analysis, the costs for various CSO mitigation alternatives are compared to the resulting benefits of water quality protection. Typically, a cost and benefit analysis will demonstrate that significant improvements in water quality protection can be attained through relatively low cost maintenance practices, especially if dry-weather overflows are a problem. Benefits to receiving water quality are estimated by using land-based and receiving water models discussed earlier.

After water quality improvements resulting from improved maintenance are achieved, the cost to provide higher degrees of protection generally increases geometrically. Typically, optimal CSO mitigation alternatives lie in the area of the "knee" of a cost-benefit curve where the marginal cost of CSO mitigation is small compared to the resulting benefit.

Once the range of solutions has been narrowed, detailed evaluation of a small number of CSO mitigation alternatives should be performed at the level of preliminary designs for specific sites. Given specific sites, subsurface soil conditions, depth of construction, constraints on use

and access to the site, and environmental impact, mitigation featur must be considered.

During the detailed evaluation of alternatives, the means of disposi of screenings and residual solids and handling and storing chemic should be addressed. In storage and pump-back schemes, it is necess to determine the capacity and routing of a force main and pump station to de-water the storage tank.

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CHAPTER 11

DESIGN OF STORMWATER IMPOUNDMENTS

I. INTRODUCTION

Stormwater impoundments are flood flow detention or retention facilities. The larger ones are similar to the small dams that the Soil Conservation Service has built all over the country for the last half century. However, the majority of stormwater impoundments are smaller and have been designed for individual developments. They are intended to control peak flood flows in accordance with some local criteria (as opposed to being part of some overall drainage plan).

The design characteristics depend upon the local objectives, which may differ across the country. In addition to peak flow reduction, these objectives can include water quality enhancement, creation of open space (in urban areas), public recreation, creation of wildlife habitat, groundwater interception or recharge, and enhancement of property values.

Stahre and Urbonas (1990) list a total of nine different categories of stormwater detention that can be used to meet these various local objectives. For reasons explained later in the chapter, a system of impoundments designed for flood control can also provide a substantial degree of water quality control at little or no increase in capital cost. Stormwater management criteria therefore should normally include water quality control provisions. For best results, such facilities should be planned on a regional basis.

A. Environmental Considerations

Stormwater impoundments must be designed with adequate consideration of environmental impact. Either wet or dry basins, unless properly maintained, may become breeding grounds for mosquitoes or other insects. In some cases, destruction of large trees can cause adverse repercussions.

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One particularly serious environmental issue is preservation of wetlands. Fresh and saltwater wetlands are protected by federal law, and sometimes by state or local jurisdictions (see Chapter 2). In evaluating stormwater management interfaces with a wetlands program, it is usually not clear what specific wetlands attributes are being protected, or whether the need for protection is absolute or can be traded off for other values.

All surface impoundments can act as settling basins for suspended solids. During construction, such impoundments can often be deepened to temporarily trap sediments from the construction site. They can be restored to their permanent design function after construction has been completed.

B. Water Quality Provisions

As previously discussed in Chapter 5, stormwater from developing and developed areas characteristically contains various contaminants (heavy metals, hydrocarbons, nutrients, and bacterial contamination) in amounts that are environmentally unacceptable.

In stormwater management, water quality control may be obtained through dual-purpose basins designed, first, to reduce flood damages downstream, and second, to reduce nonpoint source pollution from storm runoff. Although similar in concept to the much earlier sediment control and flood retention programs of the U.S. Soil Conservation Service, the idea of using stormwater detention basins to reduce environmental pollution first gained currency through the Section 208 (of the Federal Water Quality Act) water quality planning studies started in 1975.

The underlying principle of dual-purpose detention is that the detention of flood flows for reduction of damages downstream, and the retardation of flood flows for settlement of particulates, can advantageously be combined in the same structure. Flood damages are almost entirely due to floods with return periods of more than two years, whereas the harmful pollution impacts occur mainly as the cumulative effect of a large number of small storms (see also Chapter 12). Storage of the runoff from storms of up to two-year frequency, for periods of 24–36 hours, in either wet ponds or extended dry detention basins, can reduce total contaminants by approximately 60% for lead and hydrocarbons, and can achieve somewhat lower removal efficiencies for phosphates and other contaminants (Stahre and Urbonas 1990). By means of longer detention times, more complex forms of dual pond detention, or special wet basins, even higher water quality control efficiencies can be obtained (Hartigan 1989).

The design of dual-purpose detention basins is in most respects the same as that of other detention basins. The main differences are with respect to outlets. The lower portions of a detention basin, uses for storing the water quality design storm, should, wherever possible, be

designed to maximize flow length from inlets to discharge points and avoid short circuiting. To keep metals bound to sediment particles, it is desirable that sediment pH be kept near 7 and that sediments remain aerobic. This more apt to be a problem in deep wet ponds.

Infiltration basins are sometimes used effectively for water quality improvement. Infiltration basins are usually built to recharge ground water, and care must be taken to avoid introducing unacceptable concentrations of contaminants. Infiltration basins are excellent from the viewpoint of instream water quality, since no sediments are released downstream.

Finally, there are circumstances in which stormwater detention basins may be built for water quality control only. Water quality needs are especially acute in coastal areas. Many small watersheds draining into tidal waters are not greatly affected by fluvial floods. Storm tides may be of predominant concern. In such cases, it may be unnecessary to require developers to provide detention storage to reduce peak storm flows. However, the seacoast, bays, and estuaries into which these streams drain are often environmentally fragile, and may be seriously affected by bacterial pollution, hydrocarbons, heavy metals, nutrients and other contaminants which the stormwater from developed areas may contain. The prolonged retention and sedimentation of runoff from small storms, such as the one-year frequency storm, will greatly reduce the adverse impact of storm runoff in such cases. The proper hydraulic design of the principal spillway can also provide temporary sediment storage. See Chapter 12 for further discussion.

II. TYPES OF IMPOUNDMENTS

The form that impoundments or detention facilities can take is almost limitless, as are their distribution, number, and size (APWA 1981; Taggart et al. 1982). However, there are two basic approaches to designing detention storage. When facilities are planned on an individual basis for the site being considered (rather than as a part of some overall regional plan), the structure may be referred to as "on-site" detention storage. The others are "regional" detention facilities (note that Stahre and Urbonas (1990) identified the two types as "source control" and "downstream storage" facilities, and that other designations have also been used).

On-site facilities usually are designed to control the short, intense storms that produce the highest peak flow, and are usually not located most advantageously for reduction of downstream flood damages. The total volume of runoff of such storms is quite small, and the detention time is relatively short. Therefore, unless design criteria require very low release rates, the effect upon flood peaks disappears rapidly downstream. Under some conditions, detention only in the lower portion of a watershed may actually increase the peak flow at the outlet of the

watershed. Under other conditions, detention storage in the upper portion of a basin appears to be less effective.

The principal advantage of on-site facilities is that developers can be required to build them as a condition of site approval. Major disadvantages include the larger total land area required compared to regional basins, and, above all, the nuisance (odor, appearance) potential due to the difficulty of ensuring maintenance of all of these small facilities.

Facilities designed as part of a watershed planning process, in which the stormwater management needs for the watershed as a whole are developed in a staged "regional" plan, are called regional facilities. They provide more storage, and are more apt to be designed for longer release periods. The main disadvantages of the regional or watershed planning approach are the complex arrangements required to collect funds from developers and to use those funds efficiently for the intended stormwater management structures. Another disadvantage of regional basins is that they can leave substantial portions of the stream network unprotected, and plans must avoid this condition.

Generally, a stormwater management plan for a watershed incorporating a regional facility can produce more economical and effective results than the numerous small detention basins that usually result when each developer of a site provides his own detention facilities (Dendrou and Delleur 1982). As noted earlier, regional facilities also offer potential benefits, such as recreation and water quality enhancement. Despite this, the practical and institutional factors are so strong that by far the greater number of stormwater impoundments built in North America are on-site, rather than regional, facilities.

The major types of stormwater impoundments are:

(a) Detention—The temporary storage of flood water which is usually released by a measured but uncontrolled outlet. Detention facilities typically flatten and spread the inflow hydrograph, lowering the peak. Structures that release storage over a period of 12–36 (or more) hours may also serve water quality purposes (State of New Jersey 1986).
(b) Retention—Storage provided in a facility without a positive outlet, or with a specially regulated outlet, where all or a portion of the inflow is stored for a prolonged period. Infiltration basins are a common type of retention facility. Ponds that maintain water permanently, with freeboard provided for flood storage, are probably the most common type retention facility.

Detention and retention facilities can be further subdivided into:

(a) On-Stream Storage—A facility that intercepts the streamflow directly. On-stream storage occasionally is provided as an on-site facility, though it is more often an integral part of a watershed or regional stormwater plan.
(b) On-site Facilities—Special attention must be given to the design of outlet structures for controlling runoff from rooftops, parking lots, and

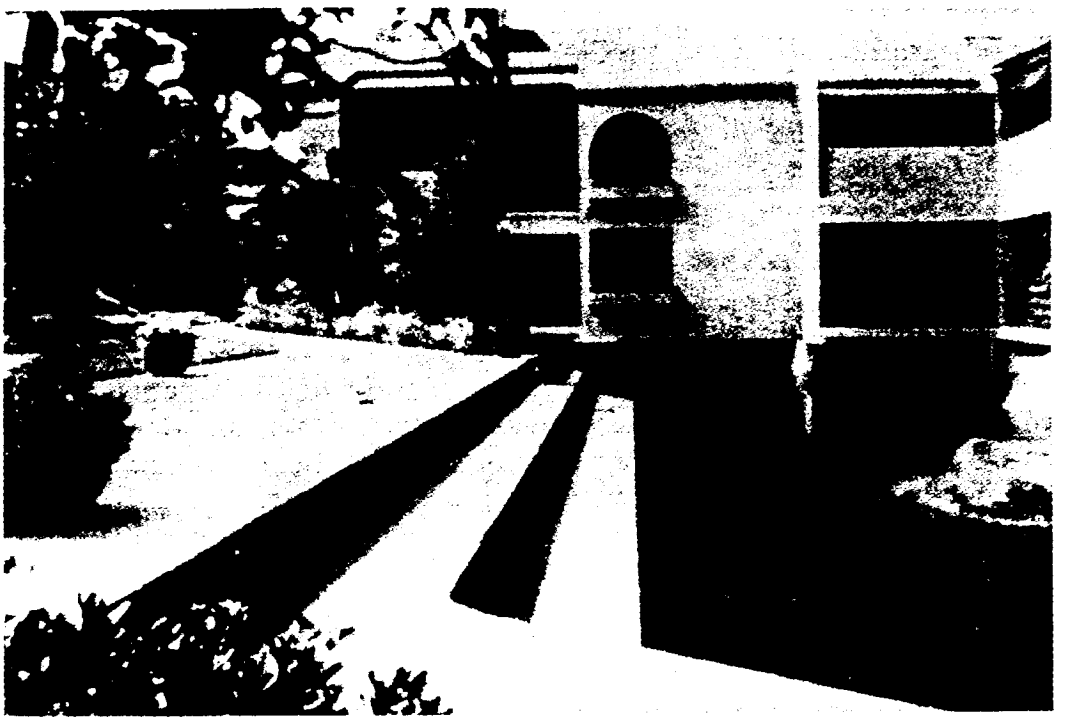
swales. Because runoff volumes from such areas are small, the required outlets are also small, which increases the potential for plugging by debris. Also, the outlet must release temporarily-stored water in a reasonable amount of time. As an example, parking lots must drain relatively fast in order not to be a nuisance. Roof top storage must be designed so as to provide safety of the structure if outlets are plugged.

(c) Off-Stream Storage—Diversion of flow out of the stream into a separate storage facility. A typical example is a side channel spillway that diverts storm flows from the stream into a storage impoundment (or a structure that can divert and store the "first flush" of particularly contaminated runoff).

(d) Conveyance Storage—Conveyance storage is an often neglected form of storage, because it is dynamic and requires channel storage routing analysis to identify. Slower-flowing conveyance caused by flatter slopes or rougher channels can markedly retard the buildup of flood peaks and alter the time response of the tributaries in a watershed.

(e) Wet Basins and Infiltration Basins—Wet basins are detention basins designed to maintain a permanent pool of water. In most aspects their design is similar to other detention basins (dry basins), except for the permanent pool. Wet basins are used for aesthetic or water quality enhancement, or for the maintenance of fish or wildlife. All outlets are above the normal level of the pool. Infiltration basins resemble other detention basins in most respects, though they may be built without

Copper Mountain, Colorado—Wet retention pond in a mountain resort area. A log wall has been provided for splash protection.



Wet detention pond on a site with limited space. Excellent example of what is possible if a landscape architect is involved in the facility design.

outlets. They may retain flood flows for a prolonged period of time, for the purpose of encouraging infiltration into the groundwater.

When evaluating the relative merits of extended dry detention versus wet detention ponds, there are several factors to consider (see also Chapter 12 for further discussion on this subject). Extended dry detention generally requires much less storage volume (land area) than wet ponds (Hartigan 1988), yet despite this, wet ponds have other offsetting advantages, which include:

(a) Wet ponds generally provide more pollutant removal.
(b) Wet ponds are usually viewed as an amenity, which will enhance property values (and offset their higher cost).

Since the water in a detention structure may cause an inconvenience to those using the land for other purposes, the temptation is great to modify or eliminate the on-site detention after construction by changing the outlet. For example, perimeter curbing that serves as the outlet control for parking lot storage is frequently damaged by owners of the lot to release water at a faster rate. This is an institutional problem that needs to be addressed locally to ensure continuous and reliable func-

tioning of such outlet facilities. Proper location and protection of such outlets can reduce these problems. Also, public education programs may be helpful. Manufactured outlets that are more difficult to alter can be used to protect roof drains.

III. THE TWO BASIC APPROACHES

A. On-site Impoundments

The typical regulatory approach to stormwater management is to require each developer to create on-site detention storage to reduce the peak outflow from his site after development, to the outflow that would occur from the same storm under pre-development conditions. Some jurisdictions require that the total post-development outflow volume not exceed the total pre-development outflow (however this is impracticable to achieve unless site conditions permit inflitration). More realistically, the criteria may specify that post-development **peak** outflow from a given storm may need to be reduced from its pre-development **peak** to compensate for the increase in volume.

When stormwater management first became common, some impoundments were designed for control of runoff from only a single return period, the 100-year storm. This sounds like an impressive criterion, but is in actuality rather ineffective, since such a design may provide little control of smaller floods, may actually increase downstream erosion, and may decrease the effect on control of larger floods downstream.

To illustrate the value of multiple-storm regulatory criteria, a comparative analysis was made of an assumed 8000 acre drainage basin in New Jersey, controlled by an on-site detention facility on each of 400 identical 20-acre watersheds (Whipple et al. 1983). The results are shown in Table 11.1.

It will be seen that Design 1, which requires only on-site control of a 100-year design flood, has virtually no effect on floods at a point downstream where the stream drains the entire 8000 acres. Design 2, which controls both 10-year and 100-year storm on-site, is somewhat better downstream, but only for large storms. The flood peaks below the frequency of 10 years, which largely determine channel erosion tendencies, are affected very little. Design 3, however, which requires on-site control of 2, 10, and 100-year floods, as well as retention of particulates, is most effective in reducing all types of floods downstream; however it also falls short in total control.

It is apparent that, in such a watershed, the requirement to control only the 100-year flood is completely ineffective a few miles downstream. On the other hand, three times as much storage is required to control peak flows for the 2, 10, and 100 year floods, and to obtain retention and settling of particulates. It is noteworthy that even with

TABLE 11.1. Comparison of Effectiveness Downstream of Various Designs.

Flood Design & Criterion (1)	Storage, Per Basin, ft^3 (2)	% Reduction in Peak Flow for 8000 Acre Watershed		
		2-yr (3)	10-yr (4)	100-yr (5)
1. Single outlet, control 100-yr flood to pre-D peak	38,000	0	0	2
2. Single outlet, control 10 and 100-yr floods to pre-D peak	89,000	1	7	30
3. Triple outlet, particulate retention and control 2, 10, and 100-yr storms to pre-D peak	113,000	44	24	20

these relatively strict criteria, the post-development peak flows at the bottom of the basin are larger than those occurring pre-development, the total volume of flood flow was considerably increased, and downstream landowners obviously do not receive adequate protection.

In another example, an investigation was made of three different design criteria; (a) regulating the 10-year flood to pre-development levels, (b) regulating the 100-year flood to pre-development levels and (c) limiting both the 10-year and 100-year floods both to pre-development levels. This analysis demonstrated that design for a single frequency, such as the 100-year flood event, has only a marginal effect on lesser events such as the 10-year, particularly as the area of the watershed increases. Design criteria addressing both the 10- and 100-year runoff can result in facilities that significantly control those events and also control increases in flood peaks further downstream (Urbonas and Glidden 1982). The engineer is strongly urged to examine a range of frequencies when undertaking any impoundment design.

The testing of criteria for on-site detention usually requires what has been called the "micro-macro" approach. In this approach, several small test areas (micro areas) representing typical subdivision and on-site detention designs are modeled. Runoff hydrographs for both pre- and post-development conditions are prepared for each micro area for the precipitation events desired. These micro hydrographs are then input to the macro model, which represents the appropriate major stream network. The advantage of the technique is that only one or a few discrete micro areas need to be modeled to identify the typical response from subdivision-type developments, with sufficient accuracy for basin-wide policy determination.

B. Watershed Planning and Regional Detention Basins

The regional approach to stormwater planning and management is generally preferable to the piecemeal site-by-site approach, though design objectives are usually not as clear-cut for watershed planning as for on-site developments (where the primary purpose usually is outflow regulation, rather than downstream flood control). It is usually economically more effective to develop a watershed or regional plan which will result in the control of storm flows in a manner equivalent to that achieved if the flows were controlled on a site-by-site basis.

Regional plans usually provide for a relatively small number of regional impoundments for flood control only, supplemented by a large number of small on-site facilities designed to retain the small floods in the interest of water quality and erosion control. The action of the larger regional impoundments makes it unnecessary to provide control of the larger floods at individual development sites. Figure 11.1 shows a watershed with three regional basins with smaller on-site facilities for water quality control. The alternative would be to require flood control storage at each of the sites shown.

The interrelated flood routings of multiple reservoirs within a watershed should be done iteratively, evaluating the relative merit (or impact) of various combinations. Deletion or addition of impoundments may point out the optimal combination. The watershed stormwater management plan may require different design criteria for different facilities. Unless an entire watershed is to be developed at one time there is no necessity to build all the master plan detention basins at once. They must, however, be carefully designed for staged construction.

Example 11-1: Tulsa, Oklahoma (Onsite vs. Regional Storage) As part of a master plan for the 3,000 acre Vensel Creek watershed prepared for the City Engineering Department of Tulsa, Oklahoma, various options for storage were evaluated (Taggart 1978). Vensel Creek (see Figure 11.2), is a tributary of the Arkansas River, with six upper tributaries which are well-defined streams in hilly wooded bluffs overlooking the Arkansas River Valley. The lower stream collects flows from the tributaries as it travels south roughly parallel to the Arkansas River. This lower stream reach is in the floodplain of the Arkansas. Its hydraulic capacity is quite limited, with overflows occurring at many points during heavy precipitation events. The overflows travel generally west to the Arkansas.

An important part of this study was to investigate regional and on-site storage options. Tulsa criteria called for regulation of developed flows from both the 5-year and 100-year design rainstorms. Thus, it was also important to test this ordinance within the context of the probable watershed response, the desired objective of limiting peak flows to historic levels, and identification of any potential for aggravating the problem.

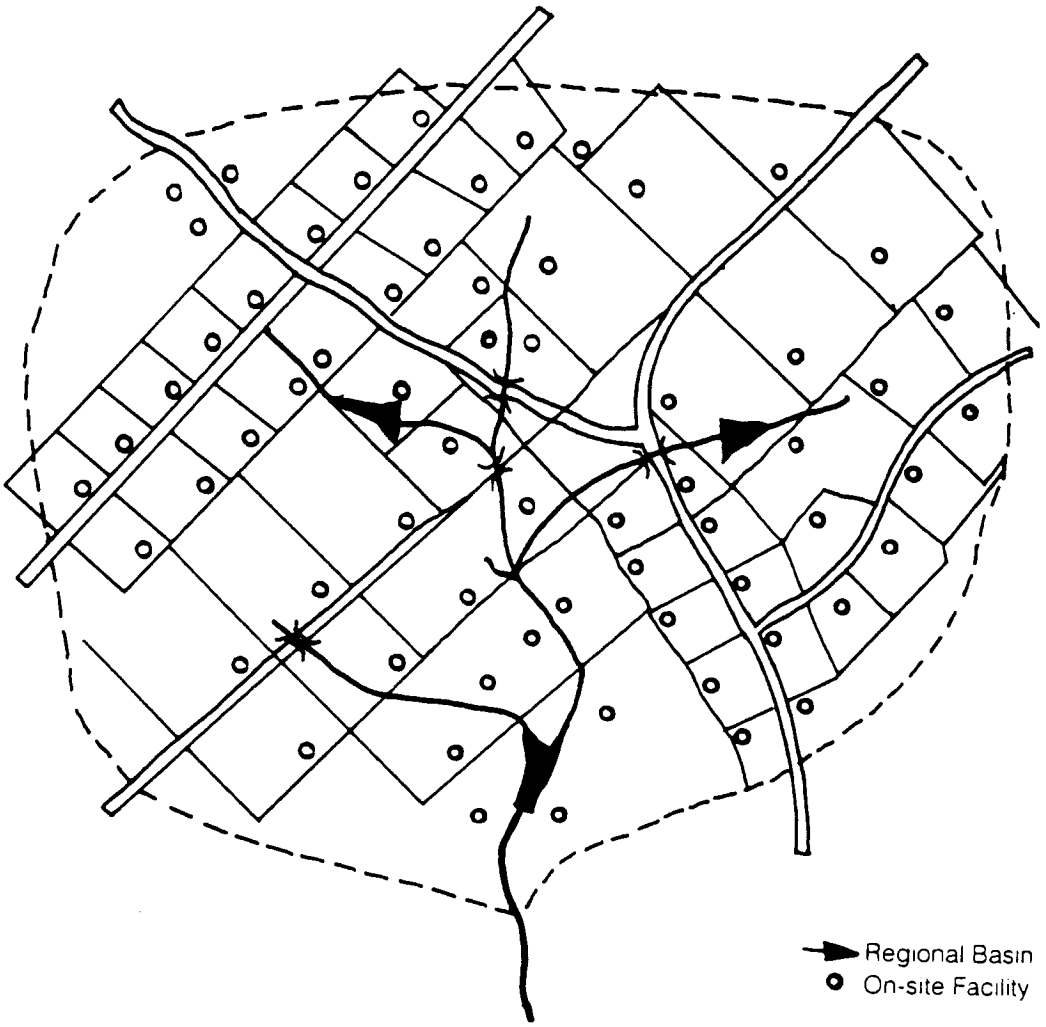


Figure 11.1—Regional detention plan.

Micro models were prepared for all the types of development proposed. These micro watersheds varied in size from approximately 8–15 acres. Detailed modeling on tributaries *E* and *H* (see Figure 11.3) allowed comparisons in basins that were largely undeveloped. Proposed development in Sub-basin *E* is largely to be conventional residential zoning with a mixture of commercial and open space, and in Sub-basin *H*, residential with lots ½ to ¾ acre in size. Detailed hydrologic analysis was performed by means of a kinematic wave model (MITCAT) that can reflect both existing and proposed conditions, including typical subdivision layout, streets, and conveyance components. For the on-site detention option, the micro model was expanded to include impoundment facilities that meet Tulsa Design Criteria.

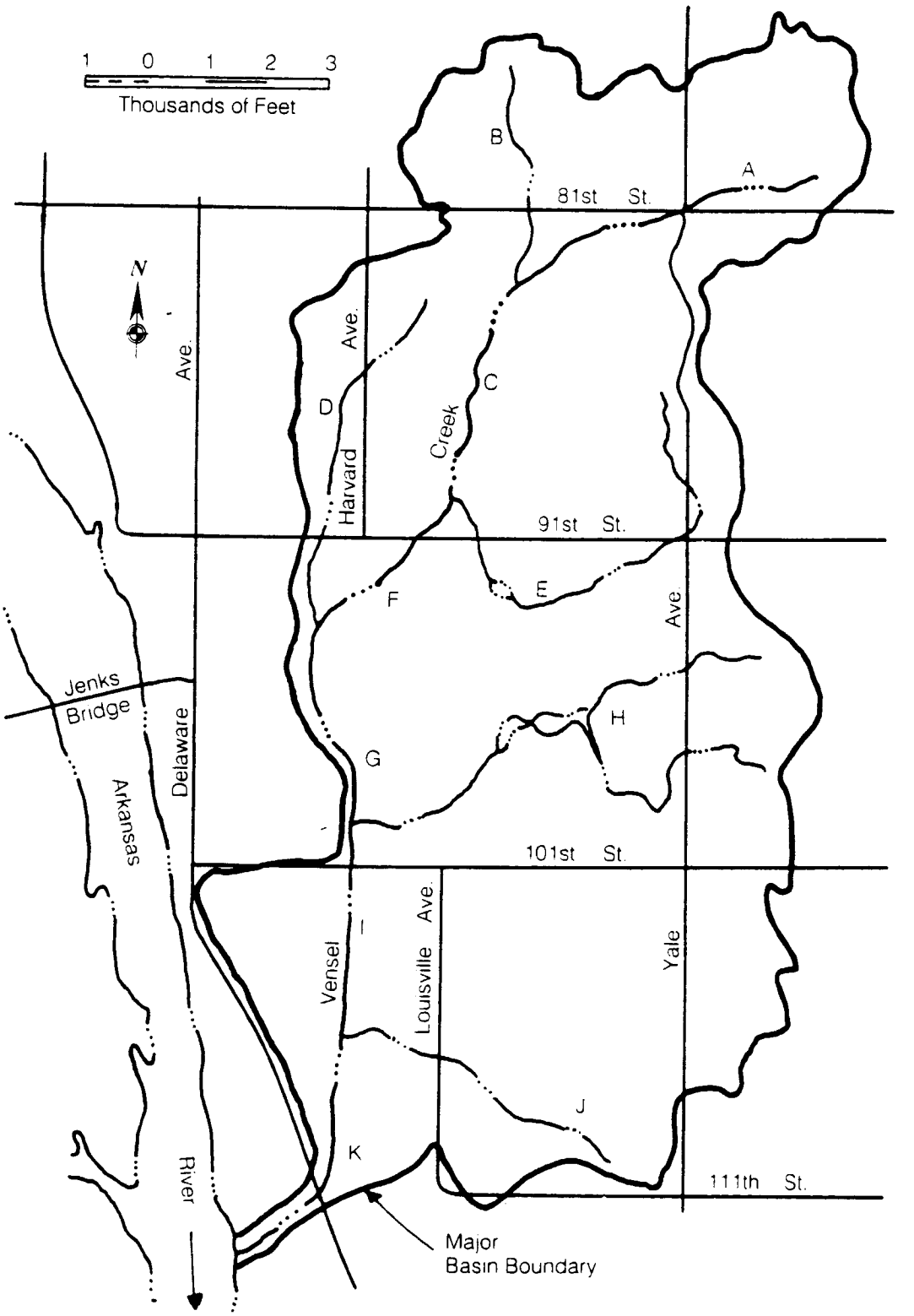


Figure 11.2—Vensel Creek drainage basin (Taggert, 1978).

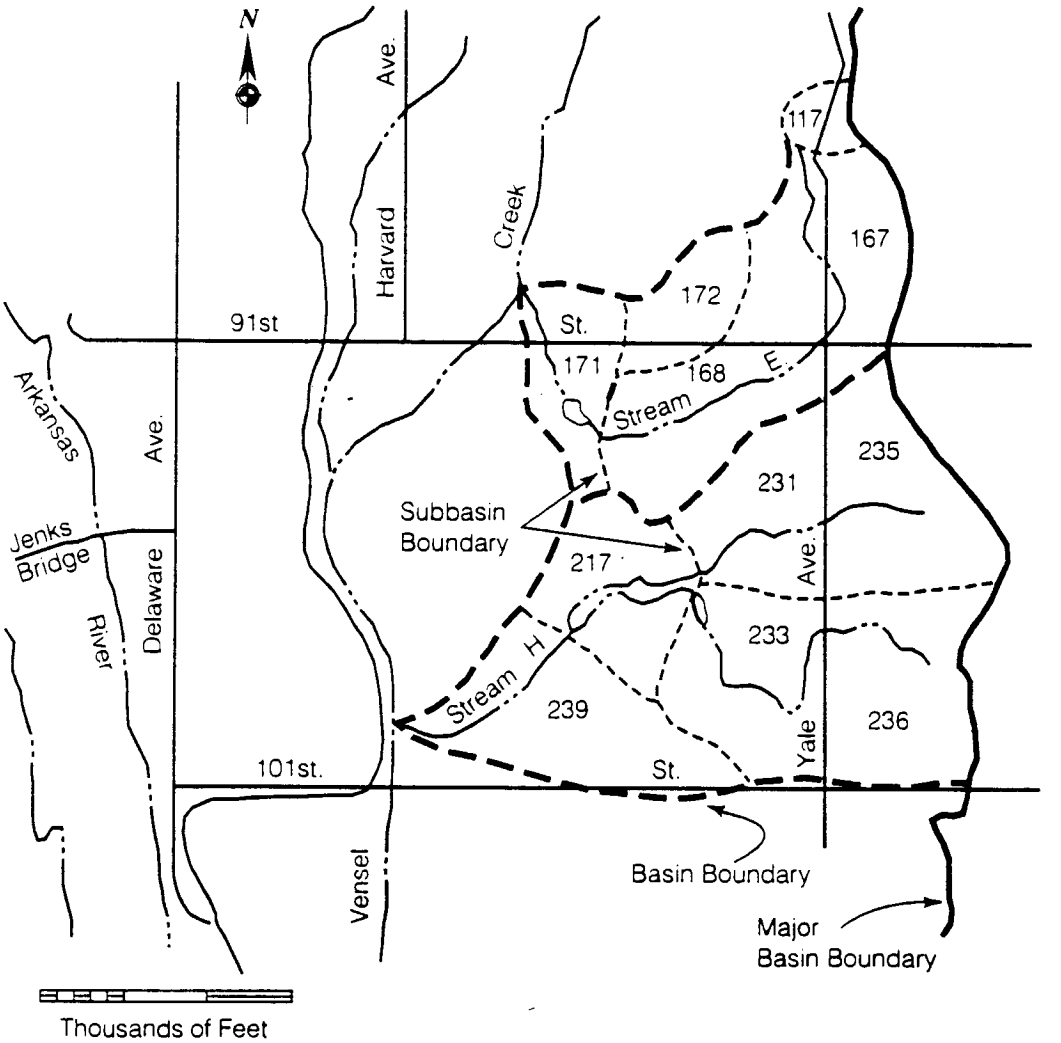


Figure 11.3—Vensel Creek basins E and H (ft × 0.304 8 = m) (Taggert, W.C., 1978).

To evaluate the overall effectiveness of the on-site detention systems, the routed hydrographs from the micro models were entered into the macro watershed model by, in effect, entering multiple micro hydrographs which would correctly reflect the total watershed area and runoff volume. Care was taken to disperse the micro hydrographs throughout the basin. For regional storage facilities, the evaluation of impoundments was simpler, reflecting storage facilities along each of the tributaries.

Table 11.2 summarizes the results for the test areas. The analysis illustrated that, in this particular case, similar storage volumes would be required for either on-site or regional systems, and that peak flow reductions are roughly comparable (note that this is **not** always the case). This particular watershed could not easily accommodate larger storage sites, so on-site facilities were favored. The reader, however,

TABLE 11.2. Summary of Vensel Creek On-site and Sub-regional Storage Test

	Sub-basin E (1)	Sub-basin H (2)
Test Areas (Acres)	287	472
100-Year Undeveloped Peak Flow (cfs)	697	780
100-Year Developed Flow Without Impoundments (cfs)	1062	1361
Sub-regional Detention System		
Resulting Test Area Peak Flow (cfs)	678	726
Storage Required Acre Feet	35	47
On-Site Detention System		
Resulting Test Area Peak Flow (cfs)	675	816
Storage Required (Acre Feet)	33	52

should not conclude that on-site facilities are more likely to provide the best solution in other cases.

IV. DETERMINING STORAGE AND OUTLET CHARACTERISTICS

While "simplified techniques" (rational method, etc.) have been widely used in the analysis and design of storage, they have many drawbacks, related primarily to the fact that the intensity-duration-frequency relationships upon which they are based were originally intended for the calculation of peak flow rates and **not** for the calculation of either the runoff hydrographs or flow volumes. The wide variety of computer models now available for calculating runoff hydrographs and storage volumes using actual rainfall distributions makes their use the method of choice for the analysis and design of detention/retention storage.

A. Rational Method

For certain small drainage areas, a detention basin sizing procedure based on the rational method may be acceptable. A simplified technique has been developed for computing the detention volume of stormwater runoff (Federal Aviation Agency 1966).

The following example, borrowed from the Urban Drainage and Flood Control Manual (Urban Drainage and Flood Control District 1984), is

based on an analysis of storm runoff at an airport where a storage cell is created by taxiways. The tributary area to the inlet is 49.5 acres, and the analysis is shown for the storms having both 5-and 10-year frequencies of occurrence.

The cumulative rainfall for the 5- and 10-year frequency was used as the rate of supply. Table 11.3 is a tabulation of the rainfall intensity in inches per hour for various storm durations, as well as the data for the cumulative runoff and the discharge for a 33-inch diameter pipe. These

TABLE 11.3. Computations for Ponding Example in Figure 11.4.

Hourly Intensities for Various Time Intervals (in/hr)

Time	5 yr. Frequency	10 yr. Frequency
5 min.	5.76	6.48
10 min.	4.92	5.70
15 min.	4.24	4.76
20 min.	3.72	4.19
30 min.	2.92	3.38
60 min.	1.87	2.28
90 min.	1.36	1.73
120 min.	1.09	1.40
180 min.	0.81	1.02

Q = CIA
A = 4.48 Acres, Pavement
= 45.04 Acres, Turf
= 49.52 Acres, Total
C = 0.90 For Pavement
= 0.30 For Turf
I = 2.00 in.
Q = 0.354 × 2.00 × 49.52 = 35.06 c.f.s.
n = 0.015 S = 0.7% 33" pipe will carry 38 c.f.s. 1 hr. = 3600 × 38 = 136,800 c.f

Distance most remote point—1600′
120′ across pavement, 1480′ across turf
Concentration Time: 4.5 + 50.5 = 55 minutes

$$\text{Average } C = \frac{4.48 \times 0.90}{49.52} + \frac{45.05 \times 0.30}{49.52} = 0.354$$

CA = 49.52 × 0.354 = 17.53
Runoff rate when all areas contributing

Cumulative Runoff in cu. ft.
I = 5.76 (From above)
Q = CIA CA = 17.53
Q = 17.53 × 5.76 = 100.97 c.f.s.

For 5 min. for 5 yr. frequency
5 min. = 300 seconds
100.97 × 300 = 30292 cu. ft.

Thus:

Minutes	5 yr. Frequency	10 yr. Frequency
5	17.53 × 5.76 × 300 = 30292	17.53 × 6.48 × 300 = 34078
10	17.53 × 4.92 × 600 = 51749	17.53 × 5.70 × 600 = 59953
15	17.53 × 4.24 × 900 = 66894	17.53 × 4.76 × 900 = 75098
20	17.53 × 3.72 × 1200 = 78254	17.53 × 4.19 × 1200 = 88141
30	17.53 × 2.92 × 1800 = 92138	17.53 × 3.38 × 1800 = 106653
60	17.53 × 1.87 × 3600 = 118012	17.53 × 2.28 × 3600 = 143886
90	17.53 × 1.36 × 5400 = 128740	17.53 × 1.73 × 5400 = 163765
120	17.53 × 1.09 × 7200 = 137575	17.53 × 1.40 × 7200 = 176702
180	17.53 × 0.81 × 10800 = 153352	17.53 × 1.02 × 10800 = 193110

(Urban Drainage and Flood Control District 1984).

data have been plotted in Figure 11.4. The discharge capacities for 21-, 24-, and 30-inch pipes are also plotted for comparison purposes.

Computations indicated that if the outlet was placed slightly more than 2 feet below the maximum pond elevation, the storage capacity would be 243,300 cubic feet. The 21-inch pipe would provide sufficient discharge to keep the maximum ponding down to 102,500 cubic feet 60 minutes after the start of the runoff for the 10-year frequency storm; however, this pipe would not empty the pond for perhaps an additional three hours.

In view of these considerations, the 33-inch pipe could be reduced in size because the smaller pipe can dispose of the ponded volume. Without the pond, the diameter of the outlet pipe would be 33 inches for the 5-year storm, and 36 inches for the 10-year storm. The results of this procedure are considered adequate for the objectives involved.

Where control of several storms of different frequency, say, 2, 10, and 100 year, is required, and for larger areas, an iterative approach is required. The designer assumes sizes for three outlets, intended to have a specified aggregate discharge. The smallest of the outlets is the lowest. Hydrographs are then prepared for post-development flows of the more frequent (smaller) storms, and a test is made of the required degree of control of each storm, starting with the smallest. A few trials will determine which combination of outlets will provide the desired degree of control of each storm. The Colorado Urban Hydrograph Procedure (CUHP), described in Chapter 5, is a good procedure for determining the effect of storage on storm runoff. The Modified Puls method described in Chapter 6 may be used for storage routing.

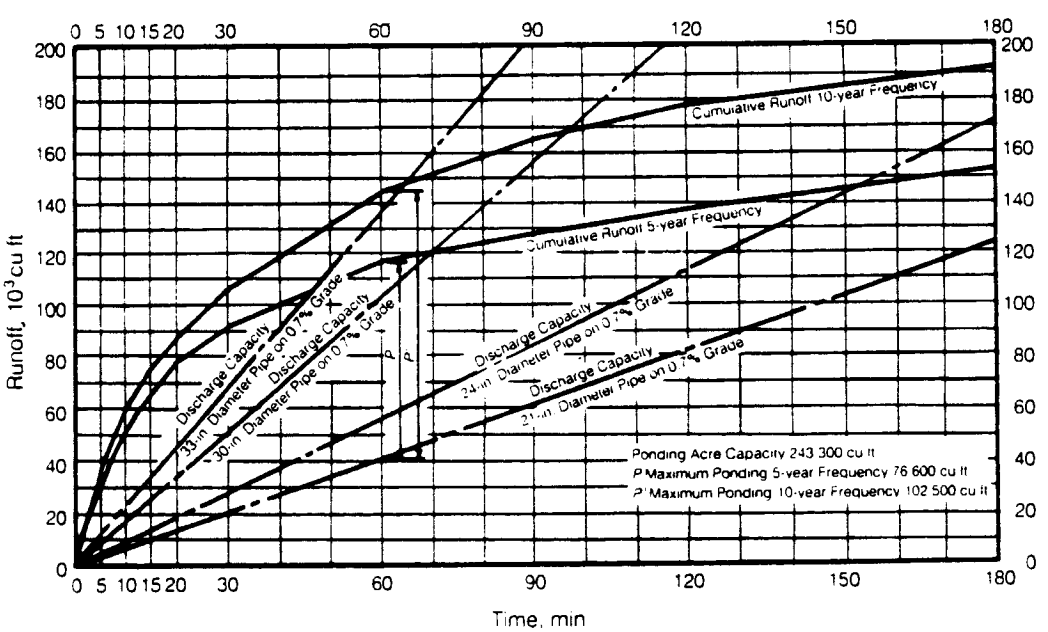


Figure 11.4—Cumulative runoff for ponding in Table 11.3 (cu ft × 0.028 32 = m^3 and in. × 25.4 = mm) (Urban Drainage and Flood Control District, 1984).

Where prolonged retention of runoff of small storms is required in the interest of water quality, a similar process is used. However, in this case, for small watersheds, the outflow from the bottom outlet is usually so small that it may be neglected. The detention basin capacity required for settlement of particulates is used to establish the elevation of the invert of the lowest flood control outlet, and design for control of floods continues as indicated above.

The above process is extremely simple, but reasonably reliable, if care is taken in estimating the coefficient C, and if good rainfall-frequency data are available.

B. Soil Conservation Service Methods

The Soil Conservation Service (SCS) has adapted the methods in Section 4 of the National Engineering Handbook (USDA 1985) to urban areas. The methods are applicable to small and large catchments. Their applicability to large catchments is based on a summation of small basin hydrographs. They are well adapted to showing differences in runoff due to differences in land use, soil type, and soil cover. For most of the United States, they are based upon use of a Type II storm of 24-hour duration, which contains within it shorter periods of more intense rainfall. These SCS methods are far more comprehensive, and cover a far wider range of conditions, than the rational method. In particular, they give volumes of runoff and complete hydrographs, rather than just peak flows (see Chapters 5 and 7 for further discussion of SCS methods).

When the entire hydrograph at a downstream location is required, it is necessary to perform a watershed routing, using the sub-catchment's runoff histogram and unit hydrograph. Space does not permit showing this calculation here. However, the method includes hydrographs derived from computer calculated watershed routings. These tabular method hydrographs can be used to develop hydrographs for a variety of commonly-occurring situations.

For various sub-catchment times of concentration, a family of hydrographs is presented. Each hydrograph represents a particular reach travel time (T_t). The hydrograph for $T_t = 0$ represents a point at the mouth of the sub-catchment. The other hydrographs are for point T_t-hours downstream from the sub-catchment mouth. This allows one to add the separate effects of each sub-catchment to obtain the total hydrograph at a particular point of interest. More specifically the procedure is:

(a) Determine the drainage area, time of concentration, and 24-hour depth of runoff for each sub-catchment (sub-basin).
(b) Calculate the travel time (T_t) from each sub-catchment to the mouth of the watershed by adding the travel times of the reaches through which it travels.
(c) For each sub-catchment, select the tabular hydrograph for the appropriate combination of t_c and T_t.

(d) Multiply these routed sub-catchment hydrographs by the sub-catchment's drainage area (in mi^2) and 24-hour runoff (inches).
(e) Add the sub-catchment hydrographs of step 4 to obtain the total watershed direct runoff hydrograph.

For single detention basins, once the flood hydrographs have been determined for pre-and post-development conditions, the pre-development flows are used to derive the allowable outflows from the proposed impoundment. The outlet characteristics then can be determined, by an approach similar to that described above for the rational method.

C. Other Modeling Procedures

There are many computer-based models (some of which are discussed in Chapter 7) suitable for the modeling of runoff and storage. The designer must choose one that is compatible with his or her capabilities and equipment, and use it to develop the inflow hydrograph(s) for the impoundment facility. The risks of working without historic precipitation and runoff data are substantial, and the reader is referred to Chapters 5 and 7 for further discussion of this important subject.

D. Provisions to Bypass Flows from Upstream

Stormwater facilities must provide for the conveyance of upstream runoff (i.e. bypass flows) while providing the necessary storage. Regulations in some areas specify a minimal bypass rate to be provided. The designer must consider the impacts of upstream runoff in both the existing and future states of watershed development. The requirement to bypass flood flows should be based on existing conditions upstream, and on the assumption that future increases in maximum discharge due to development will be controlled by the developer at that time. Provisions for bypassing of low flows through a detention facility may complicate the retention of small flood flows for water quality improvement. Spillway designs should consider full development conditions upstream in the future, since stormwater management upstream will not prevent the passage of the largest potential flood flows. If, because of particular site conditions, it is impractical for the detention basin to control all of the runoff from a development, it may be possible to control an equivalent amount of flows that not originate on-site, and that otherwise would be bypassed.

V. OUTLETS AND TRASH RACKS

A. Outlet Types

Outlets are designed for the planned release of water from the impoundment (this does not include the emergency spillway described in the following section). The Soil Conservation Service uses the term

principal spillway for the main outlet of its small dams; but the usual practice for detention basins is to refer to outlets. The outlets for detention basins are ordinarily uncontrolled (i.e. without gates or valves), and there are usually several of them. The outlets may consist of separate conduits, or several outlets may enter a chamber or manifold that leads to a single pipe or conduit (preferably from a dam safety standpoint). Outlets for detention basins must be protected by trash racks. Two excellent general references on impoundment outlet design are ASCE (1985) and Schueler (1987). The latter provides many examples of outlets that enhance the ability of impoundments to trap pollutants.

Figure 11.5 shows outlets in a dual purpose detention basin, providing water quality control features. Figure 11.6 shows a single outlet for

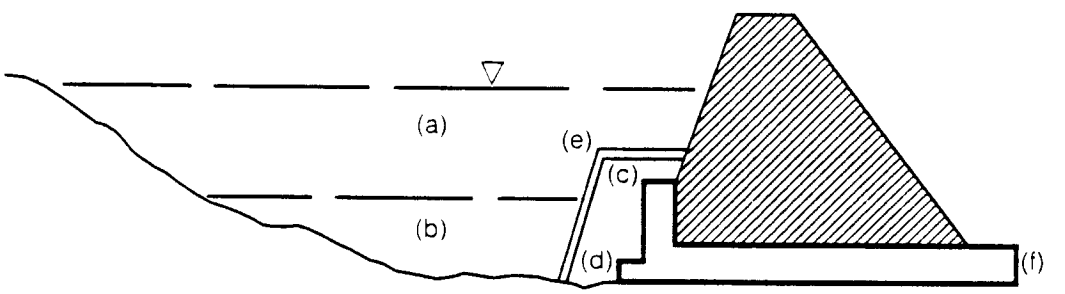


Figure 11.5—Dual-purpose detention basin outlets: (a) volume to control 100-year storm, (b) volume to control settleability storm, (c) flood control outlet, (d) water quality, (e) trash racks, and (f) downstream outlet.

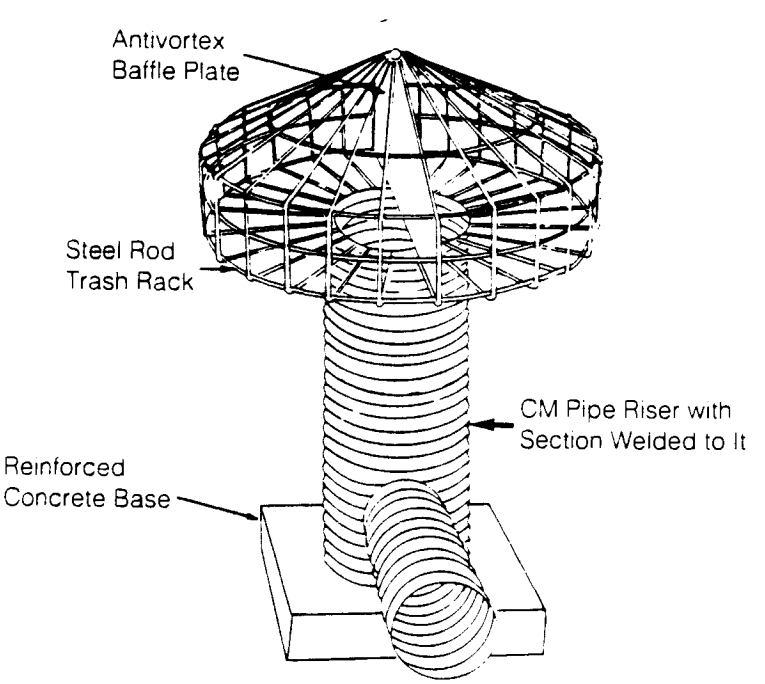


Figure 11.6—Typical wet basin outlet.

wet basins. To provide capacity at lower elevations, holes can be cut in the riser at appropriate elevations. Figure 11.7 shows a multiple stage outlet structure for a dual-purpose detention basin. Figure 11.8 (ASCE 1985) shows inlets for several types of outlet structures.

The water quality outlet will usually be small in comparison with other outlets, and a restrictor plate containing an orifice designed to provide the required outflow capacity should be used in place of a small pipe in the structure. The restrictor plate can be removed to facilitate maintenance. The capacity computation is made by the usual orifice formula.

The capacity of other outlets is determined by weir, orifice, or pipe formulas, depending upon the design. In a design such as that of Figure 11.7, the discharge of the second stage outlet would be determined by an orifice formula, and that of the third stage by a weir formula.[1] The outlet conduit is sized to carry all discharges. In this design, the overflow grate is designed primarily as a relief outlet, in the event of clogging of the others, although it would also serve to add somewhat to spillway capacity.

Where conduits are intended to function under hydraulic head for prolonged periods of time, they should have seepage-drainage diaphragms, or geotextiles should be used to control erosion of fines. Where outlet conduits discharge into easily eroded materials, stilling basins or other energy-absorbing devices should be used. In the case of on-stream overflow dams, a notched spillway may function as an outlet as well as an emergency spillway (see also Chapter 14 for other dam safety considerations).

B. Trash Racks

The susceptibility of inlets to clogging by debris and trash needs to be considered when estimating their hydraulic capacities. In most instances, trash racks will be needed. Trash racks must be large enough that their partial plugging will not adversely restrict flows reaching the control outlet. No universal guidelines exist for the design of trash racks to protect urban stormwater detention control outlets, although a commonly used "rule-of-thumb" is to have the trash rack area at least ten times larger than the control outlet orifice. For very small outlets, an even larger ratio is usually necessary to control the onrush of debris at the onset of a storm, and a high degree of maintenance is required.

Examples of trash racks are shown in Figures 11.6 and 11.7. The inclined vertical bar rack is most effective for the lower stage outlets. Debris will ride up the trash rack as water levels rise. This design also

[1]Rossmiller (University of Wisconsin 1990) has determined through laboratory studies that complex outlet structures (consisting of multiple sizes and shapes of orifices and weirs) do not perform in accordance with hydraulic theory, and caution is urged in the design of such outlets.

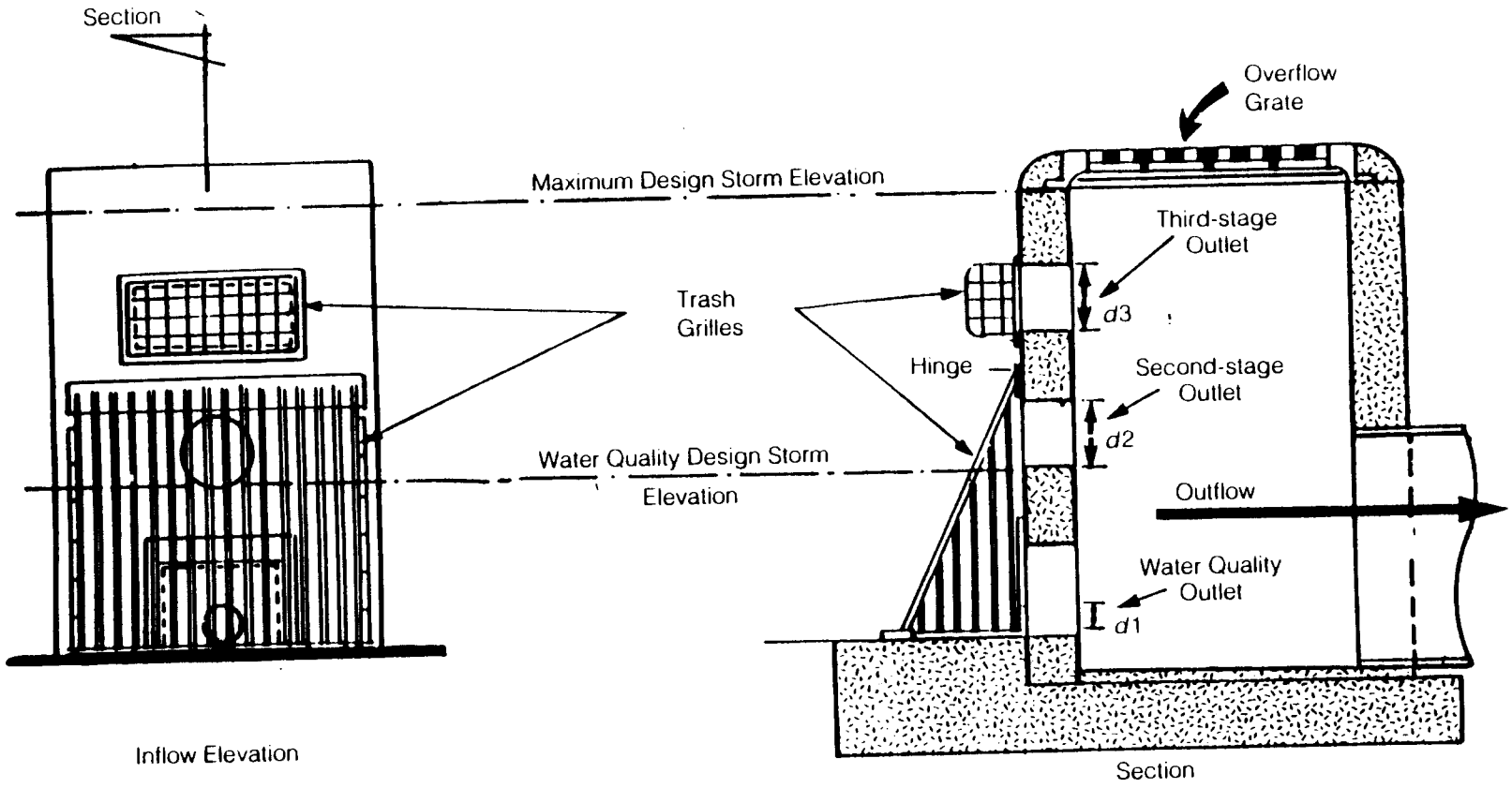


Figure 11.7—Multiple-stage outlet structure.

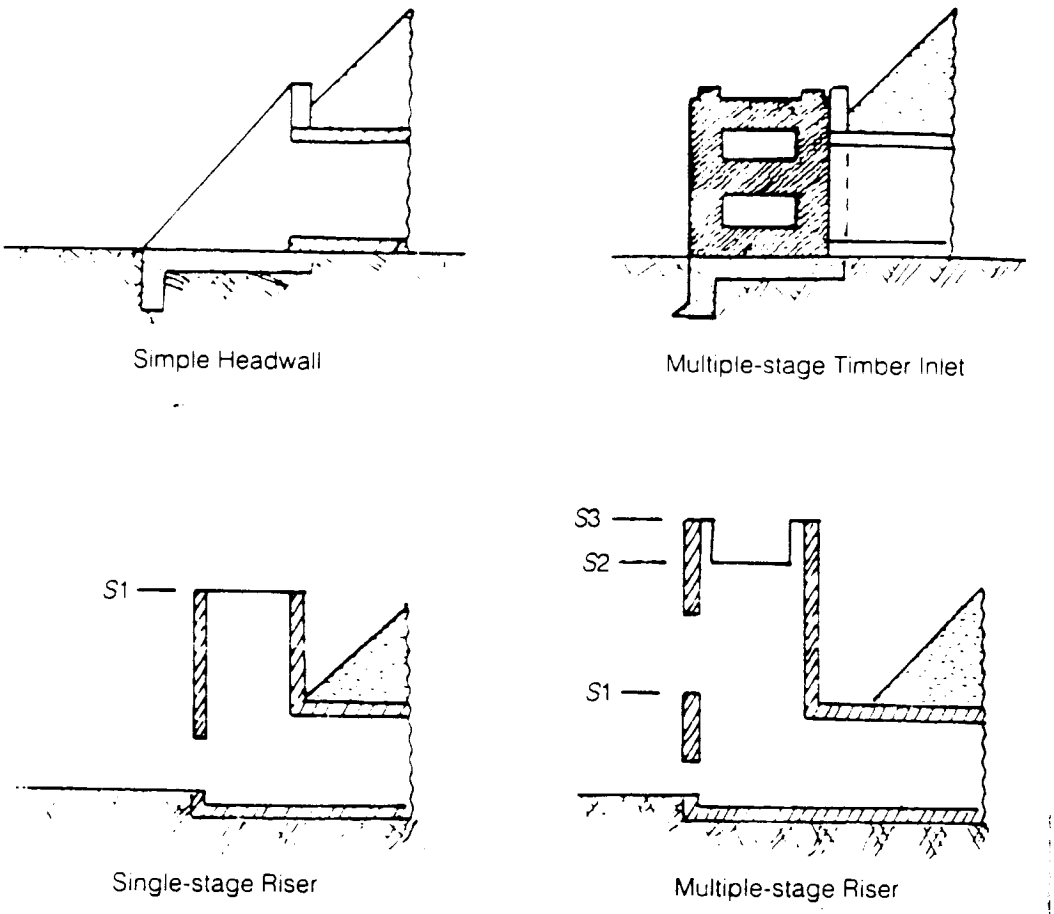


Figure 11.8—Typical inlet arrangements (for pond outlet structures) (ASCE, 1985).

allows for removal of accumulated debris with a rake while standing on top of the structure. Cage type racks or racks with horizontal members inhibit this type of debris removal.

The surface areas of all trash racks should be maximized and the trash racks should be designed to be as far away from the protected outlet as possible to avoid interference with the hydraulic capacity of the outlet. The spacing of trash rack bars must be proportioned to the size of the smallest outlet protected. Where a small water quality outlet orifice is involved, a separate trash rack for that outlet is frequently used, so that a simpler, sturdier trash rack with more widely spaced members can be used for the other outlets. Spacing of the rack bars should be wide enough to avoid interference, but close enough to provide the level of clogging protection required.

To facilitate removal of accumulated debris and sediment from around the outlet structure, the racks should have hinged connections. If the rack is bolted or set in concrete it will preclude removal of accumulated material and will eventually adversely affect the outlet hydraulics. Maintenance access, including access for heavy equipment, should be provided, as well as a means to drain the pond, if necessary.

Since sediment will tend to accumulate around the lowest stage outlet, the inside of the outlet structure should be depressed below the water quality outlet to minimize clogging due to sedimentation. Depressing the outlet bottom to a depth below the water quality outlet and equal to the diameter of the outlet is recommended (Figure 11.7).

C. Outlet Safety

Outlet safety considerations include both the safety of the structure and safety to the public. The outlet works create a potential hazard when in operation due to the possibility of a person being carried into the opening. Gratings or trash racks are often used; however, with substantial pressure a person can be forced against the grate or trash rack which can, in some cases, be worse than being carried through the conduit. To mitigate this, low entrance velocities at the trash rack are recommended. Fencing or other effective measures also should be provided to exclude people from potentially hazardous areas. Such measures include site grading, planting of thorny shrubs, or grading to assure "safety ledges" along the pond perimeter.

Outlet works can be designed to reduce the hazard to the public where heavy recreational use is anticipated. For instance, a vertical riser of concrete, timber, or steel can have a series of small openings (12" or less) from top to bottom with sufficient total area to cause low velocity at the entrances. The top of such risers can be grated, or even closed. In some instances the outlet works can be fenced. Fences are not universally recommended because of maintenance and operational needs, and because most fences do not fully prevent access. Signs are sometimes used to warn the public of the safety hazards involved at the outlet works.

During the periods of no operation, there is little hazard at most outlet works, although they can be attractive nuisances during operation. The designers need to be aware that an owner can be held liable, if an accident occurs, for having created an attractive nuisance. Design of outlets for which the orifices or weirs are not accessible from the embankment or shore when functioning is one method of reducing the hazard to playing children or curious adults, but this may complicate maintenance. Pipe openings on the upstream face of the embankment, where the pipe is only partially submerged, may be fenced on three sides to inhibit access from the embankment.

Finally, although not related to failure from overtopping, designers must recognize that a substantial percentage of embankment failures are due to inadequate outlet works design and construction. Chapter 14 discusses the subject in detail. The designer is reminded to direct special attention to the following:

(a) Avoid potential piping of water along the outside of the outlet conduits by using drainage-seepage diaphragms, or by careful material selection and good compaction around the conduit.
(b) Minimize the number of conduits through the embankment.

(c) Ensure against leaky joints within the embankment.
(d) Do not use thin-walled conduit through the embankment without a protective exterior encasement.
(e) Where reasonable, design the pipe to operate under little or no internal water pressure.
(f) Provide a safety factor in outlet works openings to account for debris collection. Spillway and outlet entrances are natural locations for debris buildup. Design the pond to minimize debris migration to both. Provide for vehicle and crane access so that debris can be removed from a basin when in operation.
(g) Do not depend upon human intervention to operate gates or other controls during a storm runoff event. Gates can jam or become inoperable when needed during emergencies. People can be unavailable or diverted by other activities.

VI. SPILLWAYS, EMBANKMENTS, AND UNDERDRAINAGE

A. Emergency Spillways

In many cases, stormwater detention structures do not warrant elaborate studies to determine spillway capacity. While the risk of damage due to failure is a real one, it normally does not approach the catastrophic risk involved in the overtopping or breaching of a major reservoir. The drainage areas of many sites are so small that only very short, sharp thunderstorms are apt to threaten overtopping or dam failure, and such storms are very localized. Also, capacities of the impoundments are usually too small to create a flood wave.

By contrast, regional, on-line facilities or smaller, onsite dams with homes immediately downstream, may pose a significant hazard if failure were to occur, in which case emergency spillway considerations are a major design factor. The engineer must characterize the potential for loss of life or property damage early in the design effort.

Dam safety awareness has improved nationwide as the result of analyses and surveys by the U.S. Army Corps of Engineers. In most states spillways of even small dams must provide a specific degree of protection by safe passage over the spillway of a specific frequency flood, usually some fraction of the Probable Maximum Flood (PMF), or the 100-year flood for small structures in undeveloped localities. Criteria for undeveloped localities should be applied with prudence, as flood plains uninhabited today may be largely developed in a few years time.

The Soil Conservation Service has provided general guidance on minimum spillway capacity for non-agricultural impoundments as shown in Table 11.4 (U.S. Soil Conservation Service, undated). It should be noted that this is **not** a national standard, it has been adjusted for New Jersey conditions and is provided simply to indicate that such tables exist. It is not recommended by ASCE, and the authors of this manual suggest that the criteria not be used without careful analysis.

TABLE 11.4. Emergency Spillway *Minimum* Design Storms (Non-Agricultural) (From U.S. Soil Conservation Service, undated)—Adjusted for New Jersey Conditions

Raised Water Height (ft)[1] (1)	Total Storage (acre-ft)[2] (2)	Drainage Area (acres) (3)	Emergency Spillway Design Storm[3] (4)
≤5	≤50	≤20	10 yr 24 hr Type II or III
5.1–20	≤50	≤20	100 yr 24 hr Type II or III
≥20	≤50	≤20	100 yr 24 hr Type II or III
≤5	≥50	≤20	50 yr 24 hr Type II or III
5.1–20	≥50	≤20	100 yr 24 hr Type II or III
≥20	≥50	≥20	100 yr 24 hr Type II or III
≤5	≤50	21–320	25 yr 24 hr Type II or III
5.1–20	≤50	21–320	100 yr 24 hr Type II or III
≤20	≤50	21–320	100 yr 24 hr Type II or III
≤5	≥50	21–320	50 yr 24 hr Type II or III
5.1–20	≥50	21–320	100 yr 24 hr Type II or III
≥20	≥50	21–320	100 yr 24 hr Type II or III
≤5	all	≥320	50 yr 24 hr Type II or III
≥5	all	≥320	100 yr 24 hr Type II or III

(1) Measured from the lowest point on the downstream toe of the dam to the emergency spillway crest or, in the absence of an emergency spillway, the top of the dam.
(2) Measured below the crest of the emergency spillway.
(3) Any pond for which a state stream encroachment permit is required must use a minimum design storm of 100 yr 24 hr Type II or III for the emergency spillway.

In many instances, the small size of a detention pond virtually precludes a PMF spillway. In those cases, means to mitigate the adverse effects of overtopping of the embankment can include:

(a) Flattening the downstream embankment face.
(b) Armoring the dam crest and downstream embankment face.
(c) Using regulated floodplain delineation and occupancy restrictions downstream representative of conditions without the detention storage.
(d) Providing extra channel capacity downstream.
(e) Using a wide embankment crest, such as is common with urban roads and streets (where rapid failure seldom occurs due to modest overtopping depths).
(f) Using non-eroding dam material such as rolled soil cement or concrete.
(g) Using small tributary basins, where the rate and volume of discharge involved are limited, resulting in overtopping flows of short duration and non-hazardous proportions.

If the engineer does not adopt the PMF as the spillway design flow for a sizeable structure, he should so inform the client and be cognizant of the increased liability that he and the facility's owner may be incur-

ring. The safest method available for justifying less than the PMF is an "incremental drainage analysis," which demonstrates that during the selected design, failure of the detention embankment will not create unacceptable increases in flood stage and velocity at critical downstream cross sections.

The emergency spillway is proportioned to pass flows in excess of the design flood without allowing overtopping of the embankment. Flow in the emergency spillway is open channel flow. Normally, it is assumed that critical depth occurs at the control section. For the larger structures, to avoid the possibility of an eroded channel developing in the spillway when the spillway is not lined, it is good practice to put a small concrete curb and cutoff wall in the throat as a point of hydraulic control.

Soil Conservation Service manuals (USDA 1982) provide guidance for the selection of emergency spillway characteristics for different soil conditions and different types of vegetation. The selection of degree of retardance for a given spillway depends on the vegetation. Knowing the retardance factor and the estimated discharge rate, the emergency spillway bottom width can be determined. For erosion protection during the first year assume minimum retardance.

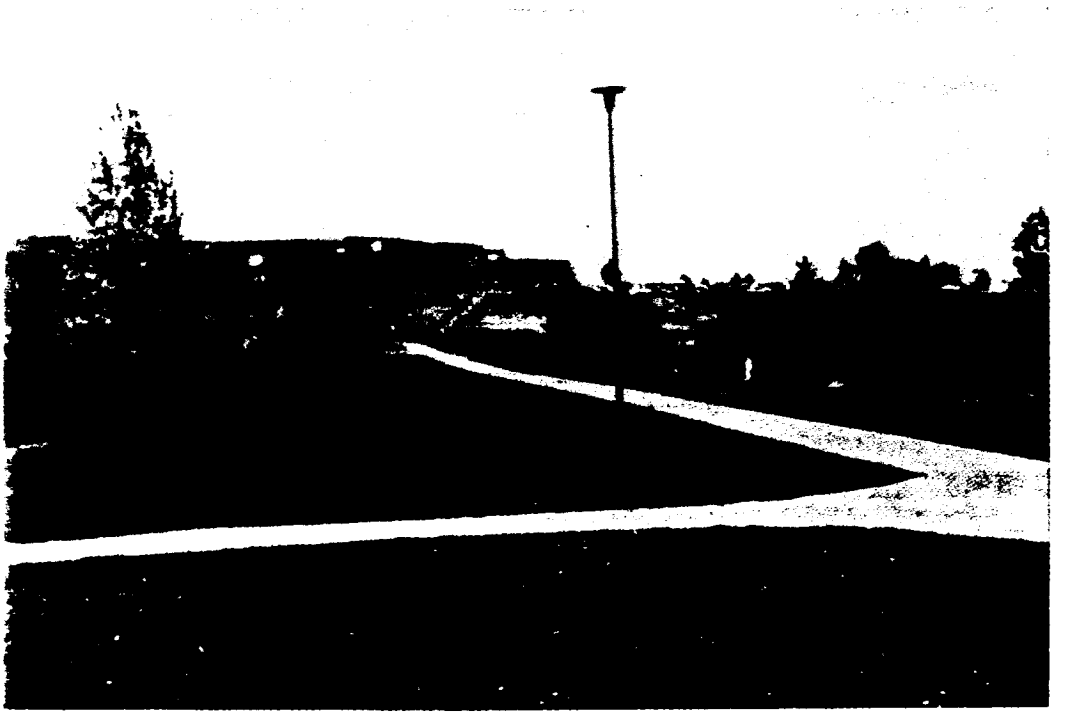
B. Embankments

An embankment that raises the water level a specified amount as defined by the appropriate state dam safety group (generally 5–10 feet or more above the usual mean low water height, when measured along the downstream toe of the dam to the emergency spillway crest), is classified as a dam. Such embankments must be designed, constructed and maintained in accordance with state dam safety standards. All other detention basins with embankments should be designed in accordance with the following criteria (which are not intended as a substitute for a thorough, site-specific engineering evaluation).

(a) Side Slopes—For ease of maintenance, the side slopes of the settled embankment should not be steeper than 4 horizontal to 1 vertical (See Table 11.5).

(b) Freeboard—The elevation of the top of the settled embankment shall be a minimum of one foot above the water surface in the detention basin with the emergency spillway at the maximum design flow. All state dam safety criteria must be carefully considered when determining the freeboard capacity of an impoundment.

(c) Settlement—The design height of the basin embankment should be increased by 10% where hauling equipment is used for compaction and 5% where compaction equipment is used. All earth fill should be free from brush, roots, and other organic material subject to decomposition. The fill material in all earth dams and embankments should be compacted to at least 95% of the maximum density obtained from compaction tests performed by the Modified Proctor method of ASTM D698.



Boulder, Colorado—Dam safety is enhanced by flattening upstream and downstream side slopes of the dam and maximizing the width of the dam crest.

Table 11.5 gives minimum recommended side slopes for earth embankments, from a stability point of view (these must be confirmed by thorough geotechnical evaluation). To provide a safety factor and facilitate maintenance, however, side slopes of 4:1 (or flatter) are recommended.

Typical dam top widths are provided in Table 11.6. The embankment, emergency spillway, spoil and borrow areas, and other disturbed areas should be stabilized and planted with appropriate vegetation. Structural analysis of earthen embankments is covered in more detail in Chapter 14.

TABLE 11.5. Generalized Side Slopes for Earth Embankments for Stability (USDA 1982).

Fill Material (1)	Side Slope Horizontal to Vertical* Upstream (2)	Downstream (3)
Clay CH Clayey sand SC Sandy clay CL Silty clay CL Silty sand SS or Clayey gravel GC Silty gravel GM	3–1	2–1
Silt ML or MH Clayey silt ML	2.5–1	2.5–1
	3–1	3–1

TABLE 11.6. Recommended Top Width for Earth Embankments (modified for urban settings) (USDA 1982)

Height of Dam (feet) (1)	Top Width (feet) (2)
Under 10	8
10 to 15	10
15 to 20	12
20 to 25	14

C. Underdrainage of Impoundment Areas

The feasibility, community acceptability, and maintenance costs of impoundments are significantly related to the facility drainage, both surface and subsurface, and the general groundwater situation in the area of the impoundment. There are many examples of facilities that can only be maintained at great expense because equipment cannot operate in soft bottom conditions, which cannot be used for recreation because the ground remains too wet, or which are generally aesthetically unacceptable.

Underdrainage or subsurface drainage of impoundments should be considered in each design case. The extent of effort and required facilities varies greatly. At one end of the spectrum, no special underdrainage improvements are necessary because depth to groundwater is large, water movement through the subsoils is fairly rapid and surface drainage in the impoundment is good. At the other extreme, however, any one of many variables can cause significant soil moisture and groundwater level problems.

1. Water Sources Contributing to Underdrainage

There are many potential sources of water that may create the need for underdrainage facilities. They include precipitation/infiltration, irrigation, utility leakage, trapped runoff due to poor surface drainage, clogged trash racks, normal streamflow via groundwater and capillary movement, flood flows via groundwater movement, general groundwater lateral movement, and general groundwater upflow. Most facility designs must consider the first five of these sources. Frequently more sources can exist, however, which can create complex interactions that require significant effort to manage. The relative magnitude of the flows involved is usually small, but if not handled properly they can effectively ruin an impoundment. The following paragraphs will discuss some typical situations and suggested management actions. For further information see U.S. Bureau of Reclamation (1978) and U.S. Department of Agriculture (1962).

2. *Surface Drainage Management*

The usual surface drainage network includes graded areas and conveyances such as swales or conduits. For general surface grading, slopes of 2% or greater are recommended, with a minimum of 1% with special subsoils and underdrainage, or where the site is acceptable as open space. For conveyance grading, a 0.75–1.0% slope is recommended. As a minimum, a 0.5% slope is acceptable with special sub-drainage facilities or when acceptable as open space, as long as adequate relief is provided.

Generally speaking, when surface drainage is less than 2%, one finds that puddling, trapped-water, damaged vegetation and conversion to nuisance plants occurs. The more frequent precipitation events and irrigation flows can leave the conveyance and large adjacent areas wet, unusable, and unmaintainable for long periods of time.

There is also water that will, regardless of the surface drainage system, continue to move by underdrainage, such as infiltrated precipitation and irrigation flow. Once these are identified they can be combined with the other subsurface sources for groundwater movement and management analysis.

3. *Groundwater Management*

It is most desirable to have the invert of the basin well above the groundwater table, and separated from it by permeable material. This is frequently not the case, and if the invert of the basin is within 3–5 feet of the groundwater level, sub-drainage facilities will be required unless the designs specify a permanently or periodically inundated pond bottom. Figure 11.9 illustrates a situation where an impoundment is to be constructed within a few feet of the existing groundwater table. At the uppermost end of the basin it may be desirable to lower the groundwater table sufficiently to prevent wet and unstable soils. A sub-drainage line placed above the barrier layer can intercept a portion of the groundwater flow.

Irrigation, which is common in the western states, can contribute significantly to groundwater problems. Numerous techniques are available to help manage this source (USBR 1978).

Frequently, impoundments have streams that flow through or along them. These flows can contribute to or create high groundwater table conditions and/or wet unstable soils. Figure 11.10 illustrates a valley cross section showing a stream, the invert of the adjacent impoundment and an intermediate embankment which might be used in an off-stream storage scheme. In this particular situation, the basin invert is 2′ above the typical stream flow water level. With the highest groundwater table shown, the groundwater will migrate toward the stream. Obviously, the depth to the groundwater table will be unacceptable. Agronomists and agricultural engineers regard 3–5 feet as a minimum depth to

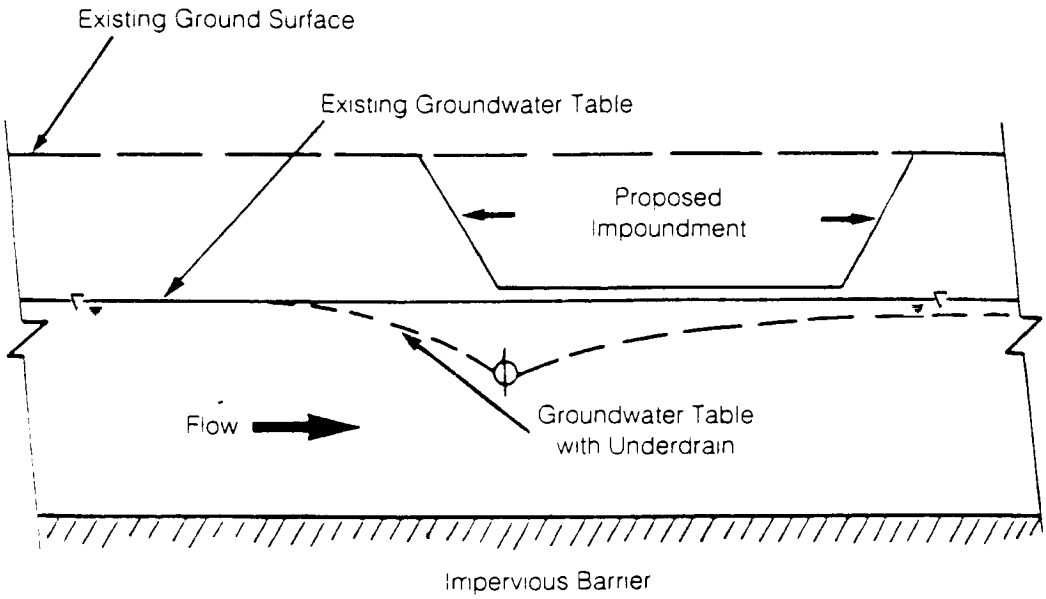


Figure 11.9—Impoundment close to groundwater table.

groundwater for reasonable vegetation conditions, and soils engineers expect unstable conditions (pumping will occur) with less than 3 feet. Even if the stream loses flow to the groundwater system (shown by the middle groundwater profile in Figure 11.10), it is possible for the depth to the groundwater table to be too shallow, especially when considering local sources such as irrigation, precipitation or infiltration. If the impoundment invert cannot be raised, an underdrain system may be required.

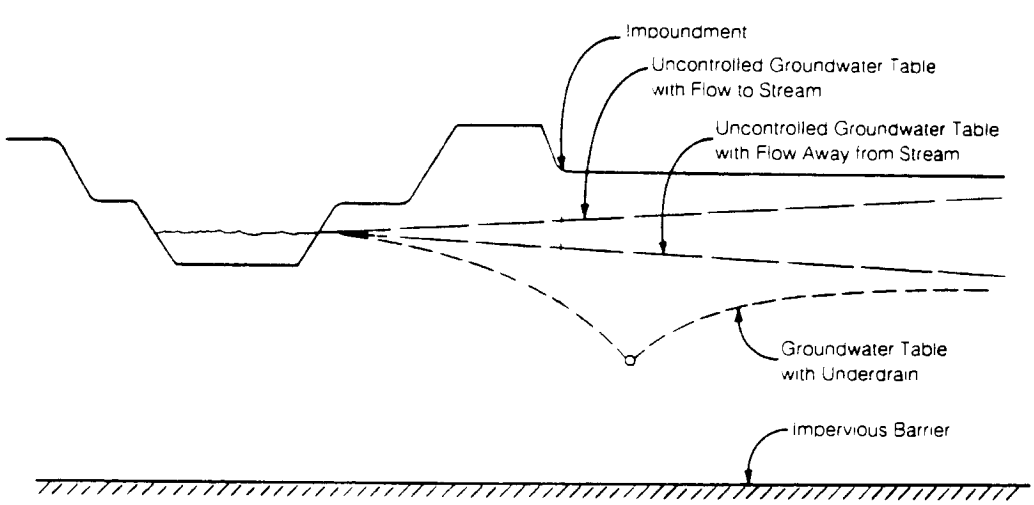


Figure 11.10—Impoundment near stream.

Finally, it is advisable to consider long-term effects of the impoundment on the sub-drainage system. Long-term sedimentation within the impoundment can form an impervious layer which could hinder the sub-drainage and surface drainage system in the impoundment.

A riser and cap may be installed at the time of initial construction for maintenance of underdrainage. This will allow attachment of the underdrain lines to a pressurized water line to flush out the underdrain lines and the drainage field. This technique, similar to back pumping on a well screen, can extend the effective life of an underdrainage system, particularly in silty soils.

VII. SPECIAL APPLICATIONS

A. Detention Basins in Flood Plains

Stormwater management and floodplain management are separate and distinct programs (see Chapter 1), each with its own rationale, but there are some unavoidable interfaces. Though desirable, it is not always possible to locate all stormwater detention structures entirely above the floodplain. Much otherwise valuable land is located within the flood hazard area, and unless it lies within the floodway, it would be inequitable to preclude its development. Therefore, the conditions must be examined under which floodplain and stormwater management objectives may be reconciled.

When land is developed in the floodplain, or only slightly above it, it is almost impossible to avoid locating the detention basin itself in the floodplain. In such a case, when the detention basin is needed to store stormwater runoff, there is a chance that the floodplain will already be flooded, and that the detention basin will already be filled by floodwater from the main valley and rendered ineffective.

Obviously, the size of the drainage area of the main channel is relevant. If it drains several hundred square miles or more, the chance of interference is rather slight, since the local flooding is more apt to come from short storms than from the prolonged or extensive general rains required to bring the main stem to flood stage. On the other hand, if the drainage area is small, the same storms will probably affect the development site and the floodplain.

In the case of the floodplain of a minor stream, a computation showing the probable effect of detention storage can be made on the assumption that the design storm, or storms, will occur simultaneously on the entire watershed. Where a design storm of 100-year frequency is used, such a computation will presumably show detention storage within the floodplain to be virtually useless for controlling that storm. Where the design storm is of higher frequency, such as 10-years, storage provided at elevations lower than those defining the flood hazard area may be effective.

B. Wet Basins

Wet basins are usually built for one of two reasons. They are frequently provided to enhance the value of adjacent properties fronting on the resulting lake, or for the improvement of water quality. They are designed to be aesthetically pleasing, with curving shapes and even islands. The engineering design of embankments, outlets and spillways of wet basins is similar to that of any other detention basin, except that the bottom of the basin is below the level of the lowest outlet, with appropriate modifications of the sides and bottom. Usually the banks require rip-rap, masonry, or gravel protection at the low water line to prevent erosion, though vegetation stabilization is sometimes adequate. The depth should be at least six feet, if aquatic plants are to be discouraged. If aquatic plants are to be encouraged, depths of less than 2–4 feet are required. Goldfish or other small fish have been used for mosquito control.

The removal of stormwater pollutants in a wet basin is accomplished by a number of physical, chemical, and biological processes. Gravity settling removes particles through sedimentation with the removal rates directly related to the pond's geometry, volume, residence time and the size of the particles. Flocculation occurs when heavier sediment

Dallas, Texas—The "hard edge" approach to design. Safety of small children is an obvious concern, although the wall is attractive.

Orlando, Florida—Good wet pond design, with flat slides slopes and a safety ledge at the interface between the ground and the water. Emerging vegetation in the littoral zone will be attractive and will enhance water quality.

In addition to being aesthetically pleasing, fountains can aid in maintaining suitable dissolved oxygen levels and in reducing algae blooms.

particles overtake and coalesce with smaller, lighter particles. The opportunity for particle contact increases as the depth of the permanent pool increases. Biological removal of dissolved stormwater pollutants includes metabolism by microorganisms that inhabit the bottom sediments, and by aquatic plants, if these are allowed to grow.

Additional pollutant removal occurs during the relatively long quiescent period between storms. The permanent water pool reduces runoff energy and provides a habitat for aquatic plants and algae—the biological filter that removes dissolved nutrients and metals. Aerobic conditions at the bottom of the permanent pool will maximize the uptake of dissolved pollutants (nutrients, metals) by bottom sediments and minimize release from the sediments into the water column (Yousef et al. 1985).

Designing wet detention systems to achieve a specified level of pollutant reduction is very difficult. Two approaches typically have been used to develop design criteria. One method relies upon solids settling theory and assumes that all pollutant removal is due to sedimentation (Driscoll 1983). The other approach views the wet detention system as a "lake" which achieves a controlled level of eutrophication, thereby causing biological, physical, and chemical assimilation of stormwater pollutants in addition to sedimentation (Hartigan and Quasebarth 1985). The basis for this design method is that stormwater is detained within the permanent pool long enough to produce adequate levels of nutrient uptake by algae and aquatic plants, but the hydraulic residence time is not so long as to induce stagnation, thermal stratification, or anaerobic bottom sediments.

Wet ponds should be shallow enough to minimize the risk of thermal stratification but deep enough to assure that algal blooms are not excessive. A mean depth of 3–10′ normally achieves these conditions and has been shown to be effective in reducing stormwater pollutants (USEPA 1983).

A shallow littoral zone is an important component of the wet detention system, since aquatic plants within this zone provide biological assimilation of dissolved stormwater pollutants. The littoral zone should cover at least 30% of the pond's surface area and slope gently (6:1 or flatter) to a depth of 2′ below the control elevation (Schueler 1987; ASCE 1990).

The littoral zone should include a variety of native aquatic plant species suitable for various depth ranges, nutrient assimilation or aesthetic purposes. The zone must either be planted with appropriate aquatic plants or covered with a 4–6 inch layer of topsoil containing the viable seeds of wetland plants. A combination of mulching plus planting one-third of the littoral zone is very cost effective and will provide a vigorous biological filter fairly quickly.

The long term viability of the biological filter is essential for good pollutant removal. Routine maintenance will include mowing along the

pond perimeter, transplanting desirable wetland species into areas needing plants, and removal of undesirable aquatic plants such as cattails. Cattails will crowd out other more desirable aquatic plants, and also deposit a large amount of vegetative matter, which decays and creates anoxic conditions and poor water quality.

Pond geometry has a very strong influence on how effectively the detention system will remove pollutants, especially in systems with a short hydraulic residence time. Little or no pollutant removal occurs in dead storage areas where the inflow is bypassed without mixing. To avoid dead storage areas, the length to width ratio should be at least 3:1. In addition, the outlet structure should be located to maximize travel time from the inlet to the outlet. The effective length of a pond can be increased by the use of diversion barriers such as baffles, islands, or a peninsula within the pond.

C. Infiltration Basins

An infiltration basin needs to be shallow with a low rate of surface application. The seasonally high groundwater table should be located at least 4′ below the bottom of the basin. Similarly, bedrock should also be located at least 4′ below the bottom of the basin. Otherwise, unacceptable ponding may take place, resulting in grass kill and insect nuisances.

The soil permeability, or final infiltration rate, will determine how rapidly the stormwater will infiltrate the ground. Soil classes with final infiltration rates of at least 0.52″ per hour (New Jersey Department of Environmental Protection 1989) allow for acceptable drain times, provided that the criteria for depth to high water table and bedrock are satisfied. In addition, soil porosity above the seasonal high water table needs to be considered. Design for a two-year runoff to fill less than one-half of the unsaturated soil mantle should provide a reasonable chance of keeping the basin from backing up and failing. Note that one foot of water depth in the infiltration basin becomes more than three feet in the soil mantle when the soil has a porosity ratio of 0.33. The water that infiltrates through the basin develops a temporary groundwater mound which drains slowly in the horizontal direction due to the very small available hydraulic gradient. This is the reason that a large storage depth in an infiltration basin has often been associated with its failure.

The soil textural class with a final infiltration rate of 0.27″ per hour (silt loam) may have some limited suitability for a very shallow infiltration basin. Soils with infiltration capacities that allow them to infiltrate 36″ of stored runoff in less than a 3-day period are particularly well suited for an infiltration basin.

Infiltration basin design should avoid the introduction of pollutants that violate groundwater quality criteria or standards.

1. *Water Table, Bedrock, and Groundwater Conditions*

Concerns related to the development of a groundwater mound below the wet pond or infiltration facility, as well as the potential for polluting down-gradient groundwater supplies, often arise when infiltration facilities are considered. Based on a limited data base for stormwater impoundments and infiltration facilities, groundwater pollution does not appear to be a problem with most residential and commercial land uses. Under many conditions, the addition of groundwater by means of detention basins is highly desirable.

2. *Runoff Filtering*

Grease, oil, floatable organic materials, and settleable solids should be removed from runoff water before it enters the infiltration basin. These materials take up storage capacity and reduce infiltration rates. Runoff filtering devices such as vegetative filters, sediment traps, and grease traps can be used to remove objectionable materials. A modified basin design such as that illustrated in Figure 11.11 can be used to enhance and prolong the infiltration capacity of the basin. When a runoff filtering system or structure is included, its design must allow adequate maintenance.

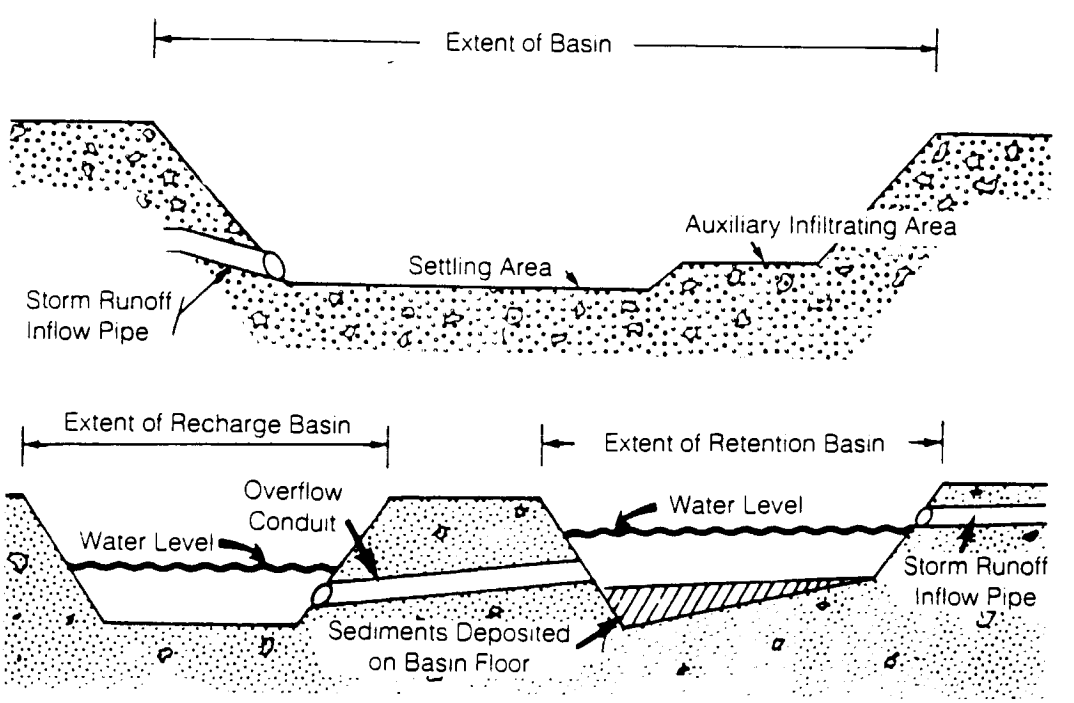


Figure 11.11—Basins to enhance infiltration.

3. *Excavation*

Initial basin excavation should be carried to within 1′ of the final elevation of the basin floor. Final excavation to the finished grade should be deferred until disturbed areas in the watershed have been stabilized or protected. The final excavation should remove all accumulated sediment. Relatively light tracked equipment is recommended for this operation to avoid compaction of the basin floor. After the final grading is completed, the basin floor should be deep-tilled using rotary tillers or disc harrows to provide a well-aerated, highly porous surface texture.

4. *Sediment Control—Vegetated Basins*

The cleanout frequency of infiltration basins will depend on whether they are vegetated or non-vegetated, and will be a function of their storage capacity, recharge characteristics, volume of inflow, and sediment load. Infiltration basins should be inspected at least once a year. Sedimentation basins and traps may require more frequent inspection and cleanout.

Grass bottoms on infiltration basins serve as a good filter material, although they may need occasional replacement. Use grass species that are most likely to work for the region of the country in which the facility is located that can withstand several days of submergence. Well-established turf on a basin floor will grow up through sediment deposits, forming a porous turf and retarding the formation of an impermeable layer. Grass filtration would work well with long, narrow, shoulder-type depressions (swales, ditches, etc.) where highway runoff flows down a grassy slope between the roadway and the basin. Grass planted on basin side slopes will help to prevent erosion.

5. *Sediment Removal From Non-vegetated Basin*

Sediment should be removed only when the basin floor is completely dry, after the silt layer has mud-cracked and separated from the basin floor. Equipment maneuverability and precise blade control are essential, and can greatly reduce the quantity of material to be removed. All sediment must be removed prior to tilling, which should be done at least once annually.

6. *Side Slope Maintenance*

Side slopes should have a dense turf with extensive root growth, which enhances infiltration through the slope surface and prevents weeds from gradually taking over. Grasses of the fescue family are recommended, primarily due to their adaptability to dry sandy soils, drought resistance, hardiness, and ability to withstand brief inundation. The use of fescues will also permit long intervals between mowings. This is important due to the relatively steep slopes which make mowing difficult. Mowing two to three times a year is generally satisfactory.

D. On-stream Impoundments

On-stream impoundments involve the use of the natural valley as the storage basin and the stream channel as the inflow-outflow conduit. Generally, an earth embankment is built to store the flood volume, although overflow structures are also used. Multiple-outlet spillways may be used to meet requirements for control of flows with different return frequencies. Open channels may also be enlarged to serve as stormwater impoundments when enough land is available and the channel has a relatively flat gradient.

On-stream impoundments are usually built as regional detention basins, but may also be on-site facilities built for a single development. In some respects, their design is similar to that of other impoundments. There are, however, some significant points of difference, which include:

(a) The on-stream impoundment must always provide for passing low stream flows. To avoid clogging and to enhance safety, the lowest outlet should be at the level of the stream bed.
(b) As a result of the need to pass low flows, it will usually be impractical to retain small storm flows in the interest of water quality.
(c) Unless the entire watershed above the impoundment is to be controlled, the emergency spillway capacity will be greater, sometimes many times greater, than would be required for an impoundment of similar storage capacity elsewhere. In such cases, an overflow dam may be used to avoid excessive spillway costs.
(d) In areas of erodible soils, sedimentation may be excessive.
(e) Backwater effects may require acquisition of rights-of-way higher than the structure itself.
(f) For on-stream impoundments, vertical drop outlets (such as those used in detention basins) must be designed with caution, because of the need to pass low flows, and because of potential sedimentation.

It should be obvious that on-stream impoundments must be designed with care and put in a proper regional context. **It is very important to note that simplified approaches, such as are often applied for small on-site detention basins, *cannot* be used to design on-stream impoundments.**

E. Oversizing Storm Sewers to serve as Stormwater Impoundments

The oversizing of storm sewers for stormwater impoundments has a narrow area of application (though it is often used in combined sewer systems to provide in-system storage). Such facilities are normally provided when land costs are extraordinarily high. For example, oversized storm sewers serving as stormwater impoundments have been used in connection with commercial developments where parking lot and roof storage availability were inadequate or considered undesirable. Oversizing storm sewers has also been used as a means of mitigating flooding problems in urbanized areas where land was not economically available.

Other limitations may include the oversized sewers' reduced ability to transport sediments, or the lack of a definitive site plan which provides for future stormwater access to the oversized sewer.

1. *Sewer and Intake Sizing*

The required storage volume can be determined by the methods discussed previously and in Chapter 5. The ultimate sewer size will be determined after considering upstream bypass flows, available head, physical constraints, and the overall hydraulics of the stormwater system.

Particular attention must be given to the location and types of intakes (i.e. inlets, catch basins, open grate manholes, and special structures), if the sewer's storage volume is to be fully utilized. In addition to evaluating the hydraulic characteristics of the intake structures, the designer should recognize that the effectiveness of intakes during major storm events can be significantly reduced by debris.

2. *Bypass Considerations*

Upstream bypass flows can rarely be economically conveyed through an oversized sewer impoundment facility, and overland conveyance should be considered.

3. *Sediment Control*

The low velocities that normally characterize oversized sewers require a careful evaluation of sediment deposition and removal. Estimates of the types and quantities of sediment accumulation are required to ascertain future maintenance requirements, and access for periodic sediment removal must be provided.

F. Recreation and Aesthetic Uses

Impoundment areas are often designed to improve the aesthetic quality of developments. Home sites are more valuable if adjacent to a lake; and corporate headquarters and industrial facilities often feature a small lake, which can function as a detention basin or as a water source for firefighting. In favorable climates, ducks and geese or other birds will occupy even a small permanent impoundment, and fish can provide a focus of interest. While the aesthetic qualities associated with any body of water provide benefits to urban and suburban developments, they also involve certain design constraints and maintenance responsibilities. If the impoundment area is to be an aesthetic focal point, the designer should consider the water quality that can be achieved. Runoff from nearby impervious areas could be channeled around the pond to avoid having greases and oils enter the impoundment area. If upstream areas will undergo development, control measures must be installed to prevent large amounts of sediment from entering the impoundment area.

If a permanent pool is to be established it should be as large and deep as possible.

Maintenance of any impoundment area designed to enhance the aesthetic quality of a developed area is extremely important. The designer should provide for easy maintenance of any area adjacent to the pool, of the area within the normal pool elevation including the outlet structure, and of any areas that will contribute runoff to the impoundment area.

When they are not storing water, detention basins should accommodate other uses such as parking; recreation sites ranging from soccer and baseball fields to handball and volley ball courts; and other more passive recreation uses such as shaded picnic sites, trails, and park benches. Some communities allow the impoundment area to be included in the open space areas of a development when calculating the density for zoning purposes. Thus the developer does not lose the total value of this land, though he may have to reorient the site development to accommodate the impoundment area.

When using impoundment areas for other uses it should be remembered that the design must accommodate both uses. Parking areas that are used for impoundments should be designed so the water never reaches depths that would damage parked cars. If some impoundment areas are to be used for soccer and baseball fields, the soils must be such that they will quickly drain to produce a usable playing surface. Any vegetation that is planted in the impoundment area must be able to withstand the expected frequency and depth of inundation.

G. Underground Impoundments

Underground impoundments should generally be avoided, except where site conditions preclude any alternative. They are ordinarily used only for on-site facilities, though there have been some very large systems built (the Chicago TARP project, Milwaukee, San Francisco, etc.). They may consist of one or more parallel tunnels, or a deep basin decked over for some particular purpose. The primary problem is one of maintenance, including the removal of both trash and accumulated sediment. Underground impoundments should be built with adequate maintenance access, and sufficient headroom for the operation of small mechanized equipment. Stahre and Urbonas (1990) provide many practical recommendations concerning the design of underground impoundments, and the reader is encouraged to study this reference before proceeding with the selection or design of such facilities.

H. Pump-Evacuated Impoundments

The control of stormwater may require that runoff be conveyed to retention basins or temporary storage impoundments. If the required storage volume and available surface area are such that the bottom of the required impoundment is below the grade of the conveyance chan-

nel, pumping may be required (see also Chapter 9 for a discussion of pump stations). In some instances, it may be possible to drain the upper portion of the impoundment over a control structure near the invert of the conveyance channel. Pumping would then begin sometime after the flood has subsided and the conveyance channel has receded below flood levels. Factors to evaluate during the design of a pump-evacuated impoundment include:

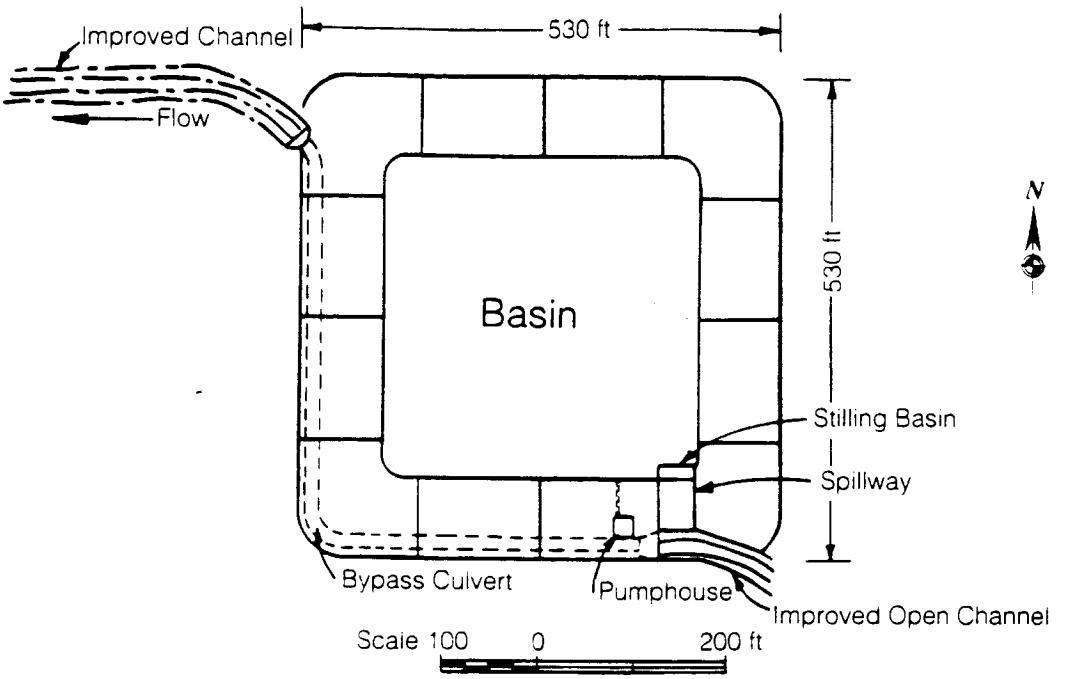
(a) Flood damage reduction versus construction and O & M costs.
(b) Required storage volume versus available surface area.
(c) Depth limitations due to adverse soil conditions.
(d) Groundwater recharge and pollution control.

At least two pumps should be installed to provide redundancy and reduce the probability of failure. An additional small pump is sometimes necessary to handle groundwater seepage. These pumps should be of the submersible type, and of a non-clogging design to allow passage of mud and small debris (see Chapter 9). A substantial concrete structure or pump wet well is usually preferred for support of the pumps. All pumps should be designed to start or stop automatically according to the wet well level or the receiving capability of the downstream channel.

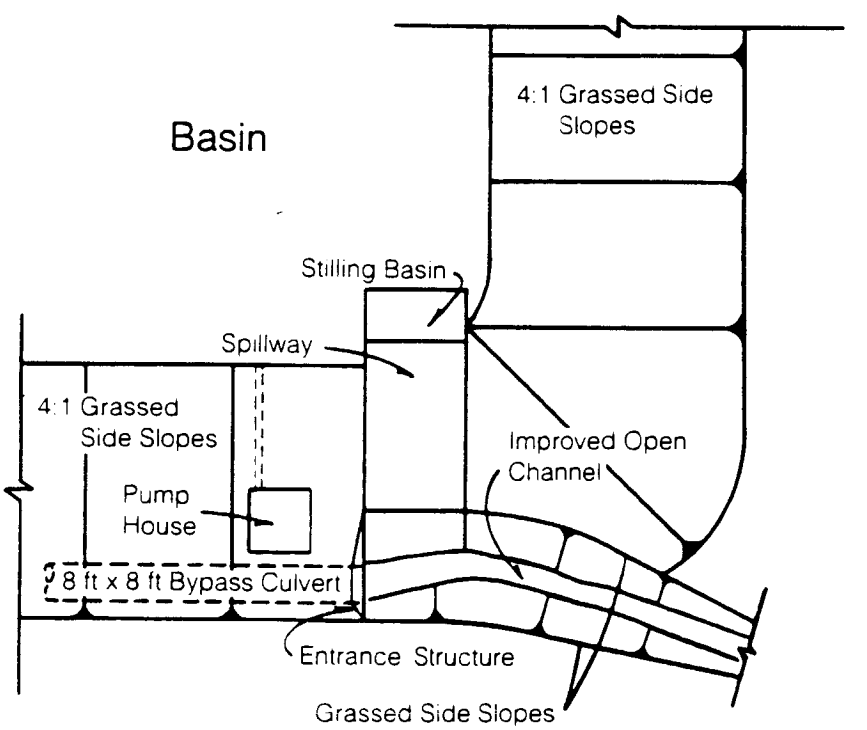
The required storage volume can be determined by developing hydrographs for selected recurrence intervals and simulating the operation of the impoundment. The difference between the inflow and the pumped and gravity outflows is the necessary storage. Several iterations may be required to determine the optimum inlet and outlet configurations. The following examples illustrate the design considerations for pumped evacuated reservoirs.

Example 11-2: Chicago, Illinois—Retention Basin to Eliminate Overbank Flooding This example is from a project constructed near Chicago, Illinois to reduce flooding from a small river in an urbanized region. The objective was to retain flood waters near their point of origin. The project, which was developed to eliminate overbank flooding caused by the 100-year storm was adopted for this comprehensive flood control plan to comply with the standards adopted by federal and local government agencies.

The recommended plan includes upstream channel improvements to increase the river's discharge capacity. A retention basin covering seven acres and with a storage volume of 110 acre feet was planned to intercept and retain a portion of the flood flows. The basin, shown in Figure 11.12, was located adjacent to the river and within the floodplain to minimize costs. By widening and deepening the channel, removing flow obstructions, and constructing a covered concrete conduit where space was restricted, the ability of the river to carry flood flows was increased. The basin invert is well below the conveyance channel, and pump-out facilities were required.



(a)



(b)

Figure 11.12—(a) Retention basin (Chicago) and (b) inlet detail (ft × 0.304 8 = m).

Flood water will flow into the basin over a concrete spillway and down a concrete lined channel to a stilling basin. For storms with less than a five-year return period, the river storage will not exceed the spillway crest elevation, and the entire flood will bypass the retention basin through an eight-foot square bypass culvert. As the river depth increases, flow in the bypass culvert will increase, up to a maximum of 390 cfs, and flows into the retention basin will be 450 cfs. During the 100-year storm 110 acre-feet of water will be stored in the basin with a maximum depth of 20′. The basin bank slopes are four horizontal to one vertical, which will allow easy access for grass cutting and other maintenance.

Evacuation of water stored in the basin for the 100-year storm will be by gravity backflow over the spillway from elevation 793 to the spillway crest elevation 791. Two manually-started pumps will empty the basin from elevation 791 to the basin bottom elevation 773 in about three days. A smaller, automatically operated sump pump will handle seepage.

The benefits from this project included a 50% reduction in the 100-year flow rate, and increased conveyance.

Example 11-3: Los Angeles, California—Pump-Evacuated Storage Facility The second example illustrates the use of pump-evacuated storage by the Los Angeles County Flood Control District for the Walteria Lake Project. In the design of such facilities, the District employs a 50-year storm which has the maximum rainfall intensity occurring on day four.

The design hydrograph for the Walteria Lake Project is shown on Figure 11.13. It has a peak flow rate of 3,000 cfs and a required storage volume of 1,057 acre-ft. A pump station with four main pumps, each of 55 cfs capacity, is used to drain the basin. After a 50-year design storm, nearly sixty hours of continuous pumping is required to empty the basin.

Figure 11.14 shows the method used to optimize pumping and storage costs. It is usually desirable to thoroughly study the costs of storage, pumping equipment, pump station construction, and operation and maintenance, to balance costs with allowable discharge requirements.

Other inflow hydrographs for different conditions may be found to have different shapes, peaking sooner and with less pronounced maximum inflow than the Walteria Lake example. The pumping rate or outflow is plotted on the inflow hydrograph and the excess of the curve about the pumping rate represents the necessary storage (see Figure 11.14). In any such installation, consideration must be given to safety of the structure in the event of flows in excess of the design flow.

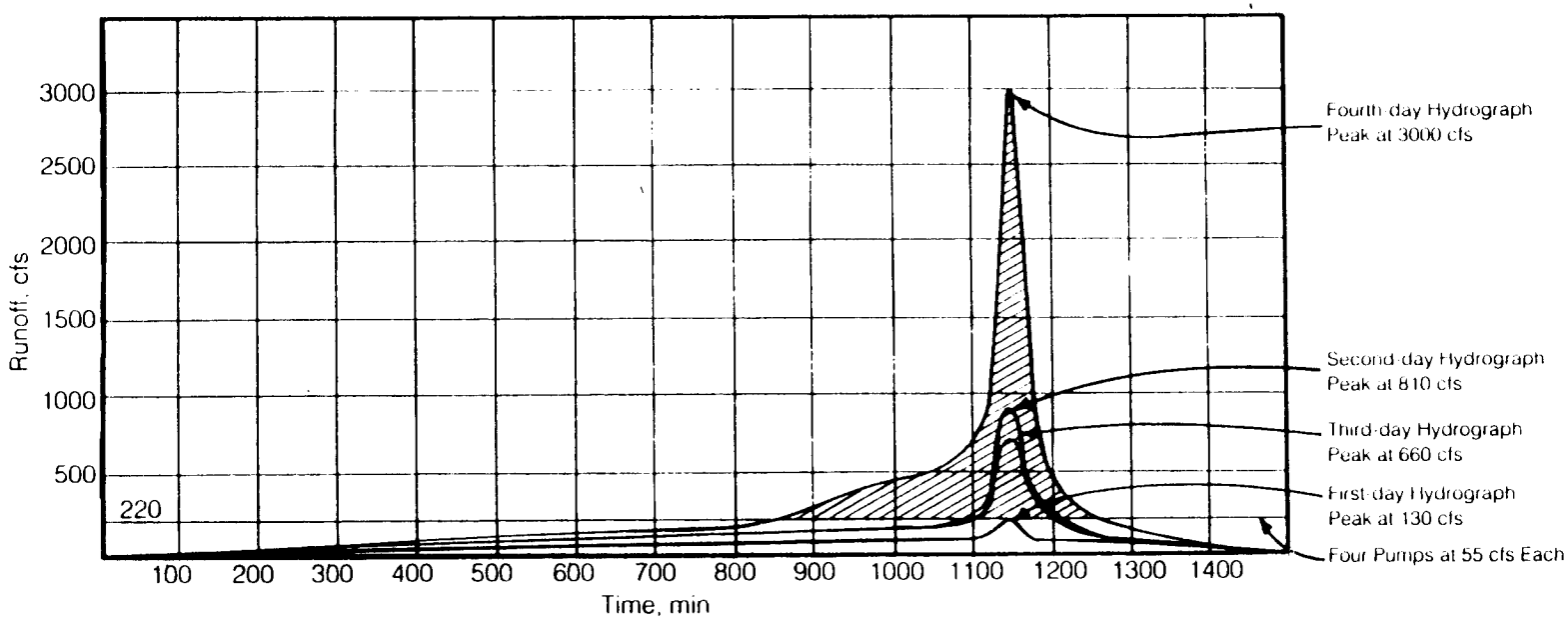
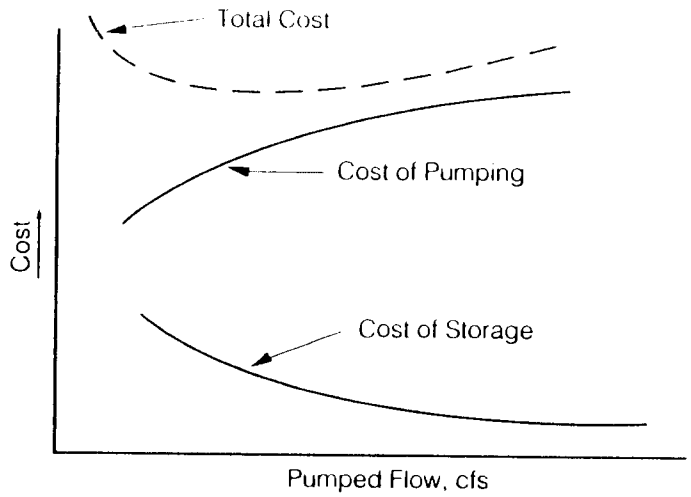
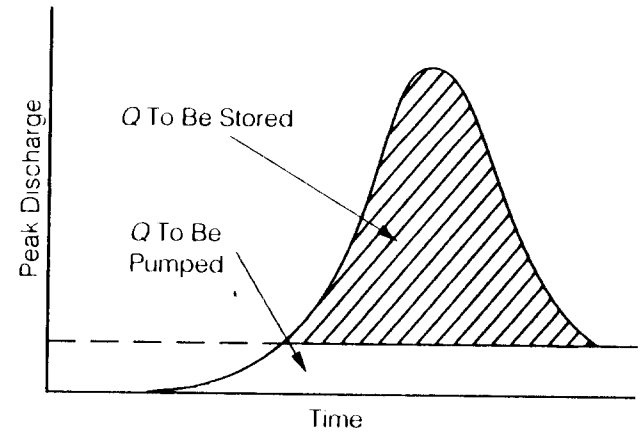


Figure 11.13—Design hydrograph, Walteria Lake project (cfs × 0.028 32 = m^3/s).



Cost of Storage and Pumping



Typical Hydrograph

Storm Day	Rainfall, in.	Runoff, in.
First	0.65	0.19
Second	2.60	0.83
Third	2.28	0.71
Fourth	6.50	2.47

50-year Storm Rainfall and Runoff

Figure 11.14—Optimization of storage and pumping costs—Walteria Lake project (cfs × 0.028 32 = m^3/s and in. × 25.4 = mm).

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IX. OPERATION AND MAINTENANCE CONSIDERATIONS

Economy and ease of maintenance are important considerations in regional planning, as well as in design of individual basins. Provisions to ensure reliable maintenance should be a key part of municipal regulations. Maintenance criteria for impoundment facilities include:

(a) Proper hydraulic functioning of the structure and its physical integrity.
(b) Avoidance of insect infestation or other nuisance.
(c) Safety and convenience of the visiting public, including children.
(d) Appearance of the facility.
(e) Utility for auxiliary functions such as recreation.
(f) Maintenance costs.

The responsibility for maintenance rests with the owner of the facility, however the local government should have the authority to inspect or review the facility to ensure that maintenance is being provided in accordance with municipal regulations.

Funding for maintenance, whether public or private, should be assured. In the case of private maintenance, two legal provisions should be made: (1) for public maintenance if the private organization fails in this responsibility, and (2) recorded easement to allow public access for maintenance. Situations should be avoided where developers sell off their building lots and depart, leaving no assurance that systems will be maintained. See Chapter 2 for more discussion of financing, legal, and regulatory issues, and Chapter 13 for further discussion on maintenance.

A. Maintenance Considerations In The Design Stage

Maintenance considerations during design include (ASCE 1985):

(a) If the detention period is long, and especially if children are apt to play in the vicinity of the impoundment, installation of an attractive fence or preferably, landscaping that will discourage entry (thick, thorny shrubs) along the periphery may be advisable. If the impoundment is situated at a lower grade and adjacent to a highway, installation of a guard rail would be in order. Access to the pond bottom and outlet is especially important.
(b) The impoundment should be accessible to maintenance equipment for removal of silt and debris, and for repair of inevitable minor erosion problems. Easements and/or rights-of-way are required to allow access to the impoundment by the owner or agency responsible for maintenance.
(c) In view of aesthetic requirements and the need for frequent mowing, bank slope, bank protection, and vegetation type are important design criteria.
(d) Permanent ponds should have provisions for complete drainage for

silt removal. The frequency of sediment removal will vary among facilities, depending on the original volume set aside for sediment, the rate of accumulation, drainage area erosion control measures and the aesthetic appearance of the pond.

(e) For basins designed for multiple-purpose use, the basin bottom needs special consideration to minimize periods of wetness. It may be advisable to provide an underground tile drainage system, if active recreation is contemplated and a flat bottom is required.

(f) Adequate dissolved oxygen supply in ponds (to minimize deterioration of water quality) can be maintained by artificial aeration. Use of fertilizer and pesticides adjacent to the pond should be carefully controlled.

(g) Secondary uses that would be incompatible with sediment deposits should not be planned unless a high level of maintenance will be provided. For example, planning a combination tennis court/detention basin may not be advisable downstream from an area prone to soil erosion.

(h) French drains and other small underground detention facilities are almost impossible to maintain, and should not be used where sediment loads are apt to be high.

(i) Underground tanks or conduits designed for detention should be sized and designed to permit entrance of mechanized equipment to remove accumulated sediment.

(j) All detention basins should be designed with sufficient depth to allow accumulation of sediment for several years prior to its removal.

(k) Wet basins should be of sufficient depth to discourage excessive aquatic vegetation on the bottom of the basin.

(l) Designing an outlet to minimize hazards is often "ensured" through construction of trash racks or fences, which may become eyesores, trap debris, impede flows, hinder maintenance, and, ironically, fail to prevent access to the outlet. On the other hand, desirable conditions can be achieved through careful design and positioning of the structure, as well as through landscaping that will discourage access (steep slopes, positioning the outlet away from the embankment, etc). Creative designs, integrated with innovative landscaping, can be safe and can also enhance the appearance of the outlet and pond. Such designs often are less expensive initially.

(m) To reduce maintenance, outlet structures should be designed with no moving parts (i.e., pipes, box culverts, orifices, and weirs). Manually and/or electrically operated gates should be avoided. The only exceptions are excavated storage or surface storage affected by tides or other high water that must be pumped dry, and structures with water-actuated flap gates. To reduce maintenance, outlets should be designed with openings as large as possible, compatible with the depth-outflow curve desired and with water quality, safety, and aesthetic objectives. One way of doing this is to use a larger outlet pipe and construct an orifice or a V-notch weir in the headwall to reduce outflow rates. Outlets should be robustly designed to lessen the chances of damage from debris or vandalism. The use of thin steel plates as sharp crested weirs is best avoided because of potential accidents, especially with children. Thin plate orifices must be protected by trash racks.

B. Maintenance Operations, General

The most important routine functions are cutting grass and weeds, removing sediment, repairing any erosion, and cleaning out debris. These operations must be planned and adjusted to local conditions, such as erodibility of upstream soils, prevalence of insect vectors, and the type of outlets. Small water quality outlets require more maintenance.

To maintain aesthetic appeal, floating debris must be removed from the pool surface after a storm. Therefore, to remove the debris, access must be provided for vehicles and boats. Accumulated sediment and weed growth can be removed by dredging or by excavating equipment after de-watering.

Where pumping facilities are required, the pump house should be designed to provide security and resistance to vandalism, which could include fencing and vandal-resistant doors and locks. Between storm events, maintenance should include lubrication and operation of the pumps on a regular schedule to ensure they will function when needed.

C. Outlet Maintenance

Outlets that are protected by a trash rack will accumulate trash during and between storm events. To facilitate outlet operation and maintenance, trash racks should be curved or inclined so that debris tends to ride up as the water level rises. Such a design leaves the rack clear and allows for easier cleaning during a storm event. The periodic removal of debris from orifice trash racks is the single most important maintenance aspect of any effective stormwater detention program.

Outlet structures may be partially or completely plugged by a build-up of deposited sediment, by floating plant growth such as water hyacinths, and by vegetation growing in the sediment. Sediment deposition is a natural occurrence in basins and periodic removal of vegetation and sediment is necessary to ensure that the intended hydraulic function of the outlet is not impeded. Such activities should be anticipated during design so that both maintenance access and a nearby waste disposal site are provided.

Proper design of the outlet structure can minimize the need for maintenance of the discharge end. High velocity outflow can erode the downstream outlet, foreslope, and channel. A well-designed and constructed energy dissipator, surrounded by large, well-graded riprap on the foreslope and downstream channel can do much to reduce the maintenance needed. Deep toe walls to resist scour (undercutting) should also be provided.

All portions of the outlet structure must be accessible to vehicles, equipment, and personnel between and during storm events. This includes the floor of the basin as well as ramps to points above the upstream and downstream sides of the outlet structure.

Provisions to remove particulate pollution are closely related to the desirability of prolonged retention of flood flows for flood damage

reduction. From the viewpoint of capital investment, the additional water quality control function is obtained virtually without cost. However, prolonged retention of storm flows for either purpose requires a small outlet, which entails additional maintenance. The grass is cut periodically as a part of general grounds maintenance. At the same time, debris should be removed from in front of the trash racks protecting small orifices. Owners of detention basins understandably prefer large orifices which impound little or no water except during exceptionally large floods. Care must be taken to ensure that maintenance personnel do not remove orifice plates in order to simplify debris removal.

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CHAPTER 12

STORMWATER MANAGEMENT PRACTICES FOR WATER QUALITY ENHANCEMENT

I. INTRODUCTION

The control of water quality in urban runoff is in its technical infancy, however there are methods and techniques to insure that the levels of pollutant reduction achieved are indeed the maximum practicable. These techniques include the control of pollutants at their source, treatment of the polluted stream, or a combination of both.

Much can be done to improve the quality of the stormwater that runs off our urban developments if the designer is simply cognizant of possibilities for maximizing water quality. Moreover, many of these controls can be worked into the project in ways that can often enhance the aesthetic value of the project rather than detract from it (Roesner 1988).

II. HYDROLOGY FOR RUNOFF QUALITY CONTROL

The design runoff (flow rates and volumes) selected for sizing water quality controls is considerably different from that used for the design of drainage facilities. The damage done to a receiving water ecosystem by uncontrolled pollutant wash-off in the 50-year event is inconsequential compared to the hydraulic damage that results naturally to aquatic habitats from such an event.

(a) Drainage systems are designed for **large infrequent** runoff events (10-, 25-, 50-, or 100-year).
(b) Design events for runoff quality control are **small frequent** events (smaller than the 1-year runoff event).

To demonstrate this, consider the graphs shown in Figures 12.1a and 12.b. The curves were produced from an analysis of 39 years of sequential hourly rainfall data in Cincinnati, using the computer program STORM (Roesner et al. 1991). Detention basins of various volumes were simulated, and the frequency and volume of overflow were computed for each size basin, assuming the basin would be drained within 24 hours after a storm event. The upper curve shows the percentage of

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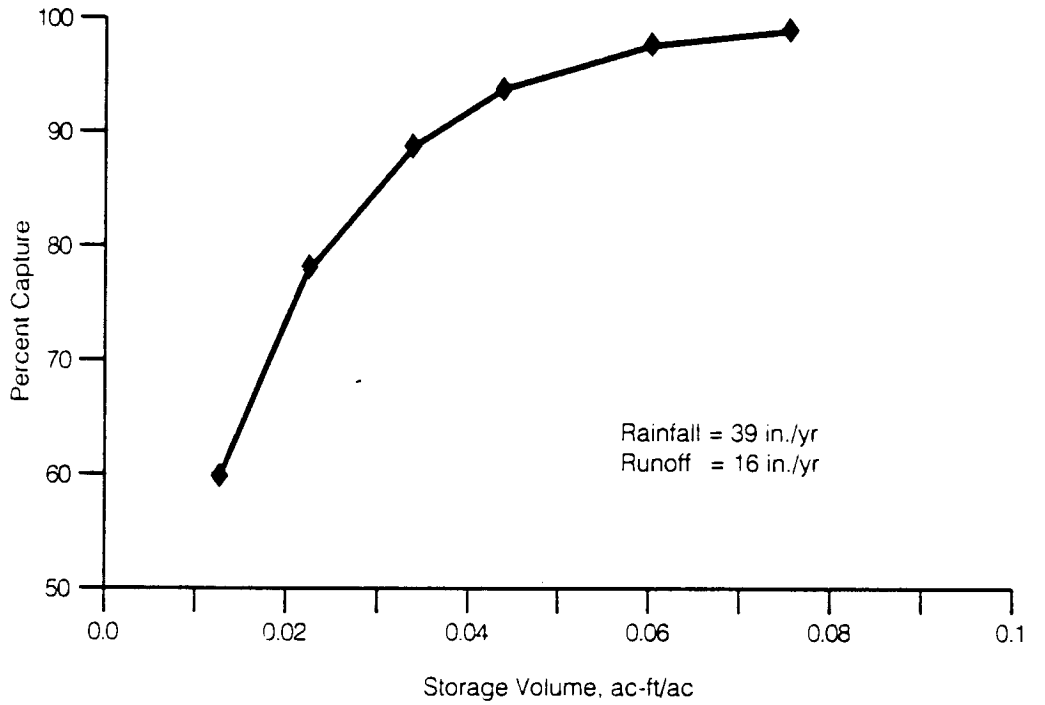
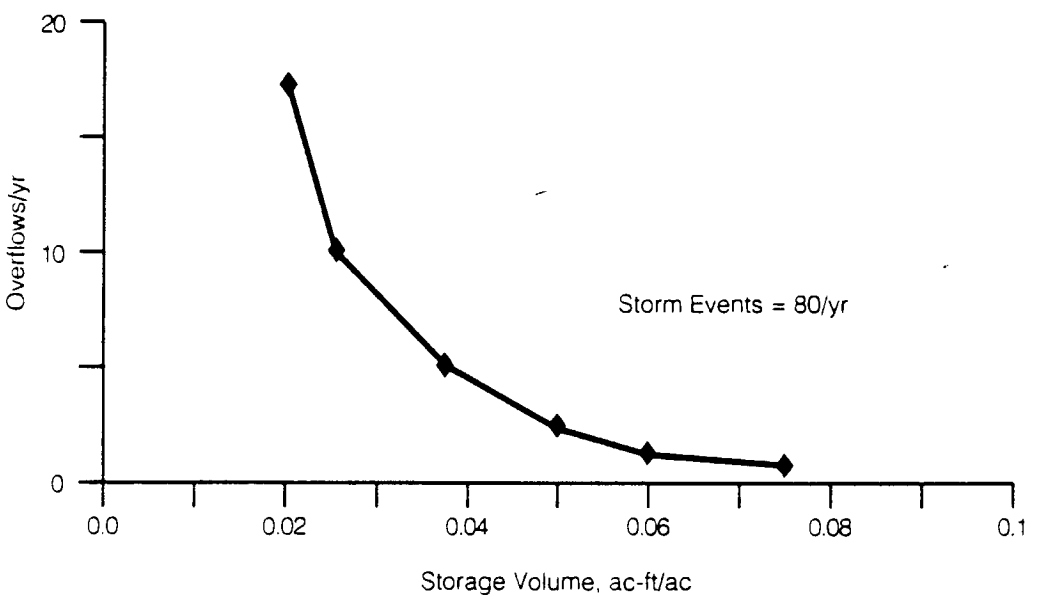


(a)



(b)

Figure 12.1—Effectiveness of surface detention on capture of runoff: (a) percent of annual runoff captured by various levels of storage, and (b) times/yr storage is overtopped as a function of storage volume (in./yr × 25.4 = mm/a and ac-ft/ac × 304 7 = m^3/ha).

the annual runoff that will be captured and detained for detention facilities of various sizes. Capturing the first 0.25 inch of runoff (0.02 ac-ft/ac) results in a capture of 75% of the runoff on an annual basis, while increasing that runoff capture to 0.5 inch (0.04 ac-ft/ac) allows capture of 93% of the annual runoff.

To see what runoff volume must be captured to attain these percentage captures of annual runoff, Figure 12.1b is used. This figure shows that, if 0.02 ac-ft/ac of storage is provided for the capture of runoff, only about 16 of the 80 events that occur per year (i.e., slightly more than one per month) overtop the detention facility. If that storage is increased to 0.5 inch (0.04 ac-ft/ac), the detention facility is overtopped only five times per year, or less than once every two months.

Put another way, capturing the 1-month runoff event (12 overflows per year) will result in capturing 80% of the runoff on an annual basis, while capturing the 2-month event (6 overflows per year) will result in the capture of more than 90% of the annual runoff.

The slope of the curve in Figure 12.1a indicates that, above a capture efficiency of about 90%, the marginal increase in storage volume required to capture another 1% of the annual runoff is very large, and thus increasing storage capacity is probably not cost-effective above this level. Application of this analysis in a number of different locations across the United States has led to similar conclusions; i.e., capturing and/or treating the first 0.25–0.5 inch of runoff (0.02–0.04 ac-ft/ac) will result in 80–95% capture of the runoff volume. However, before developing detention criteria for a given geographic area, local rainfall and reservoir routing analyses should be performed.

III. AXIOMS FOR THE DESIGN OF URBAN RUNOFF QUALITY CONTROLS

While runoff quality control is more an engineering art than a science with few established design criteria for pollutant removal, some empirical rules have been developed. They include:

(a) The most effective runoff quality controls reduce the runoff peak **and** volume (these are generally infiltration controls).
(b) The next most effective controls reduce the runoff peak (these controls generally involve storage).
(c) For small runoff events (those with return intervals of less than two years), the runoff should be retarded by detention in order to control downstream erosion (note, however, that the resulting duration of flows is longer, which can aggravate bank/channel erosion problems).
(d) Most obnoxious pollutants in urban runoff are settleable; however, appreciable amounts of nutrients and some heavy metals are dissolved and require treatment.

If these axioms are borne in mind while the designer develops the drainage plan for a development, the pollution load from the resulting system can be **significantly** reduced.

IV. SOURCE CONTROLS

Source control is a difficult matter. Homeowners generally are not restricted in the application of nutrients, herbicides, and pesticides to their yards. Chemical spillage on roadways, dustfall, etc., are also difficult to control. Nonetheless, attention to good housekeeping practices while designing the layout of the surface drainage system can significantly reduce the inevitable sources, and best management practice (BMP) programs now being introduced promise to reduce or eliminate some sources and their resultant impacts on water quality.

Source controls can also be effective at commercial/industrial sites. For example, chemical loading/unloading and storage areas could be covered or diked to capture 0.5 inch of runoff, with a closable outlet so that, if spillage occurs, it will not be washed into the drainage system before it can be cleaned up. Runoff from wash-down areas could be discharged to a sanitary sewer (though it may not be desirable if toxic chemicals are present) or to some treatment device before being discharged to a storm drain. Soil slopes should be mild (or otherwise terraced) and planted with a ground cover that minimizes erosion.

Being alert to the presence of chemicals and pollutants in areas subjected to rainfall or runoff is the first step in source control. Modifying the site plan and/or drainage plan is the second step toward eliminating these sources.

V. SITE CONTROLS FOR STORMWATER QUALITY MANAGEMENT

Site controls are generally those controls that attempt to reduce runoff rate and volume at or near the point where the rainfall hits the ground surface. The following types of site controls are common:

(a) Minimization of directly connected impervious area.
(b) Swales and filter strips.
(c) Porous pavement and parking blocks.
(d) Infiltration devices, such as trenches and basins.

Table 12.1, developed by Schueler (1987), shows the relative efficiency of various urban runoff quality controls in removing pollutants.

A. Minimization of Directly Connected Impervious Area

Directly connected impervious area (DCIA) is defined as the impermeable area that drains directly to the improved drainage system, i.e., paved gutter, improved ditch, or pipe. The minimization of DCIA is by far the most effective method of runoff quality control because it delays the concentration of flows into the improved drainage system and maximizes the opportunity for rainfall to infiltrate at or near the point at which it strikes the ground. Figure 12.2 illustrates the difference between an area where the DCIA is extensive and one where DCIA has been minimized. The residential lot on the north side of the street

TABLE 12.1. Comparative Pollutant Removal of Urban Runoff Quality Controls (Schueler 1987).

BMP/design	SUSPENDED SEDIMENT	TOTAL PHOSPHORUS	TOTAL NITROGEN	OXYGEN DEMAND	TRACE METALS	BACTERIA	OVERALL REMOVAL CAPABILITY
EXTENDED DETENTION POND							
DESIGN 1	◕	◔	◔	◔	◑	⊗	MODERATE
DESIGN 2	●	◑	◔	◑	◑	⊗	MODERATE
DESIGN 3	●	●	◑	◑	●	⊗	HIGH
WET POND							
DESIGN 4	◕	◑	◔	◔	◔	⊗	MODERATE
DESIGN 5	◕	◑	◔	◑	◕	⊗	MODERATE
DESIGN 6	●	●	◕	●	◕	⊗	HIGH
INFILTRATION TRENCH							
DESIGN 7	◕	◑	◑	◕	●	●	MODERATE
DESIGN 8	●	◑	◑	◕	●	●	HIGH
DESIGN 9	●	●	●	●	●	●	HIGH
INFILTRATION BASIN							
DESIGN 7	◕	◑	◑	◕	◑	●	MODERATE
DESIGN 8	●	◕	◕	◕	●	◕	HIGH
DESIGN 9	●	●	●	●	●	●	HIGH

KEY:

○ 0 TO 20% REMOVAL
◔ 20 TO 40% REMOVAL
◑ 40 TO 60% REMOVAL
◕ 60 TO 80% REMOVAL

POROUS PAVEMENT							
DESIGN 7	◑	●	◑	◕	◑	●	MODERATE
DESIGN 8	●	●	●	●	●	●	HIGH
DESIGN 9	●	●	●	●	●	●	HIGH
WATER QUALITY INLET							
DESIGN 10	○	⊗	⊗	⊗	⊗	⊗	LOW
FILTER STRIP							
DESIGN 11	◔	○	○	○	◔	⊗	LOW
DESIGN 12	●	◑	◑	◕	●	⊗	MODERATE
GRASSED SWALE							
DESIGN 13	○	○	○	○	○	⊗	LOW
DESIGN 14	◔	◔	◔	◔	○	⊗	LOW

● 80 TO 100% REMOVAL
⊗ INSUFFICIENT KNOWLEDGE

Design 1: First-flush runoff volume detained for 6–12 hours.
Design 2: Runoff volume produced by 1.0 inch, detained 24 hours.
Design 3: As in Design 2, but with shallow marsh in bottom stage.
Design 4: Permanent pool equal to 0.5 inch storage per impervious acre.
Design 5: Permanent pool equal to 2.5 (Vr); where Vr = mean storm runoff.
Design 6: Permanent pool equal to 4.0 (Vr); approx. 2 weeks retention.
Design 7: Facility exfiltrates first-flush; 0.5 inch runoff/imper. acre.
Design 8: Facility exfiltrates one inch runoff volume per imper. acre.
Design 9: Facility exfiltrates all runoff, up to the 2 year design storm.
Design 10: 400 cubic feet wet storage per impervious acre.
Design 11: 20 foot wide turf strip.
Design 12: 100 foot wide forested strip, with level spreader.
Design 13: High slope swales, with no check dams.
Design 14: Low gradient swales which check dams.

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has all impervious areas on the lot draining directly to the gutter. This drainage plan allows no opportunity for water falling on the impervious surfaces to infiltrate into the ground; in fact, the system is laid out so that the rain falling on the impervious areas is quickly concentrated and drained to the gutter. The result is a greatly increased peak runoff rate and runoff volume compared to the pre-development condition. The pollutants contained in the runoff from the rooftop, driveway, sidewalk and street are simply collected in the gutter and must be dealt with at some location further down in the drainage system.

In contrast, the layout for the lot on the south side of the street has been designed to minimize DCIA. All impervious areas drain to a pervious area before they reach the grassed swale that serves as the primary conveyance facility for runoff from the lot. The roof runoff drains to the lawn and flows (sheet flow) across it, the driveway is sloped to drain to the lawn instead of the street, and runoff from the sidewalk and the street flow across a grass filter strip before reaching the water in the grassed swale. All of these techniques combine to provide maximum opportunity for infiltration and for retardation of the

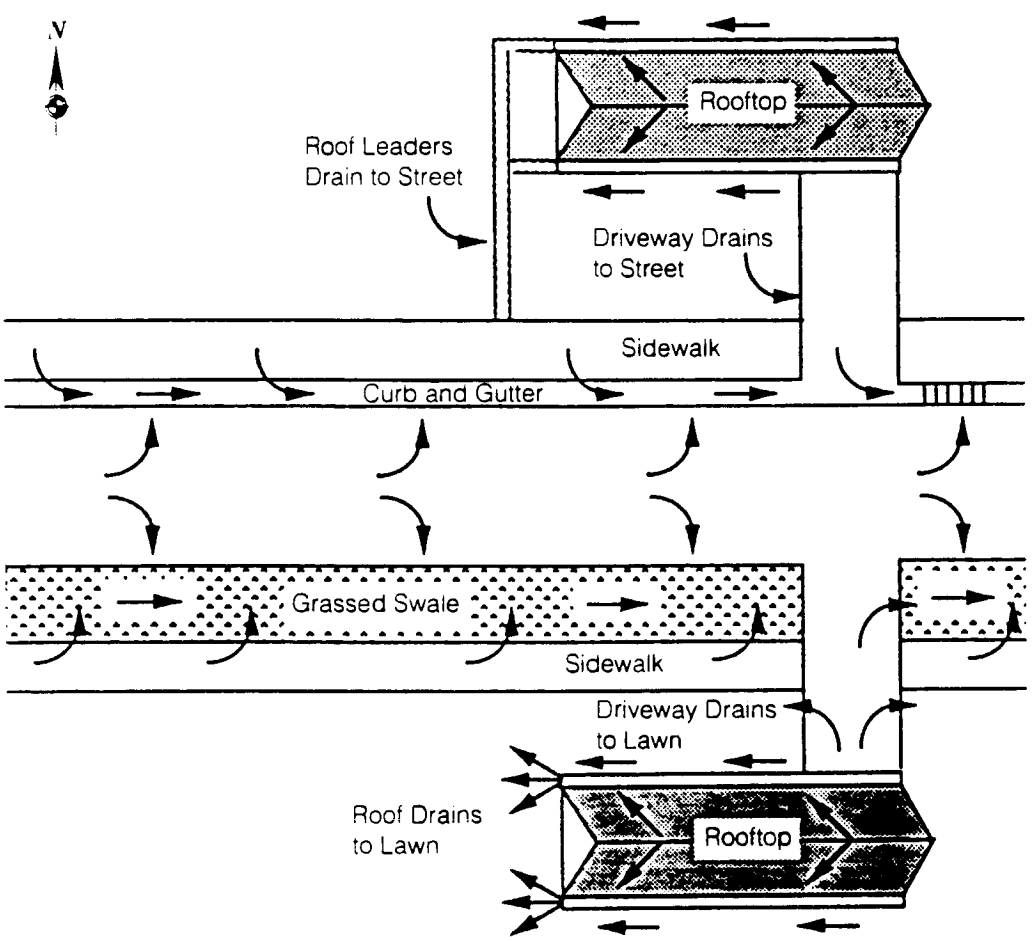


Figure 12.2—Examples of maximizing (North Lot) and minimizing (South Lot) directly connected impervious areas (Schueler, 1987).

runoff rate. This approach to drainage system layout, which emphasizes peak flow reduction and pollutant capture, is called **stormwater management**, as contrasted with the north lot design, which is simply a drainage plan.

Bear in mind that the aim of this design is minimization of the runoff peak and volume for small storms (the one- to two-month storm). The grassed swale, of course, will have been designed to transport the five- or ten-year storm through the area, so during the small events it will not be flowing full and the unsubmerged area of the swale will serve as a filter strip for the runoff from the street and sidewalks.

In the case of streets without curbs, an additional maintenance problem arises. While runoff from the street can easily flow onto bordering grassy areas, over time, as cars drive or park off of the asphalt pavement the edges begin to break up, and the grass either pushes out into the asphalt or is destroyed by the weight of the vehicles driving off the pavement. One way to minimize these problems without inhibiting runoff is to provide a one-foot-wide concrete border along the edge of the street. At intersections where cars tend to drive onto the grass to go around another car turning left, a curb can be installed that turns the radius of the corner. Figure 12.3 shows such a curbless street and intersection.

B. Swales and Filter Strips

Swales, or grassed waterways, and filter strips are among the oldest stormwater control measures, having been used alongside streets and

Seattle, Washington—Draining parking lots through grassed swales and into grassed depressions can aid in removing pollutants from parking lot runoff.

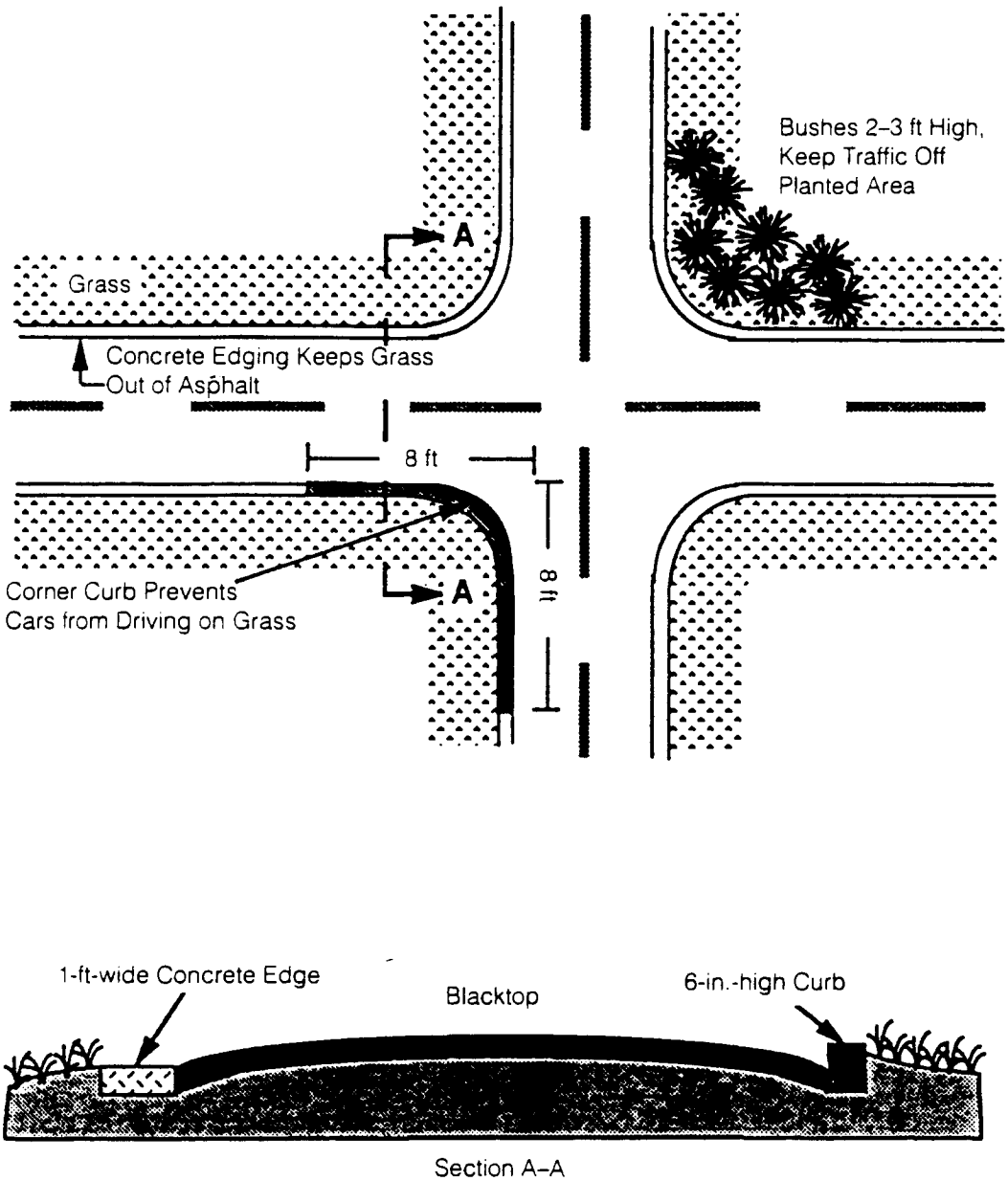


Figure 12.3—Example of concrete road edging and corner curb (ft × 0.304 8 = m and in. × 25.4 = mm).

highways for many years. A swale is a shallow trench which has the following characteristics:

(a) The side slopes are flatter than three feet horizontally to one foot vertically.
(b) It contains contiguous areas of standing or flowing water **only** following rainfall.
(c) It is planted with or contains vegetation suitable for soil stabilization, stormwater treatment, and nutrient uptake.

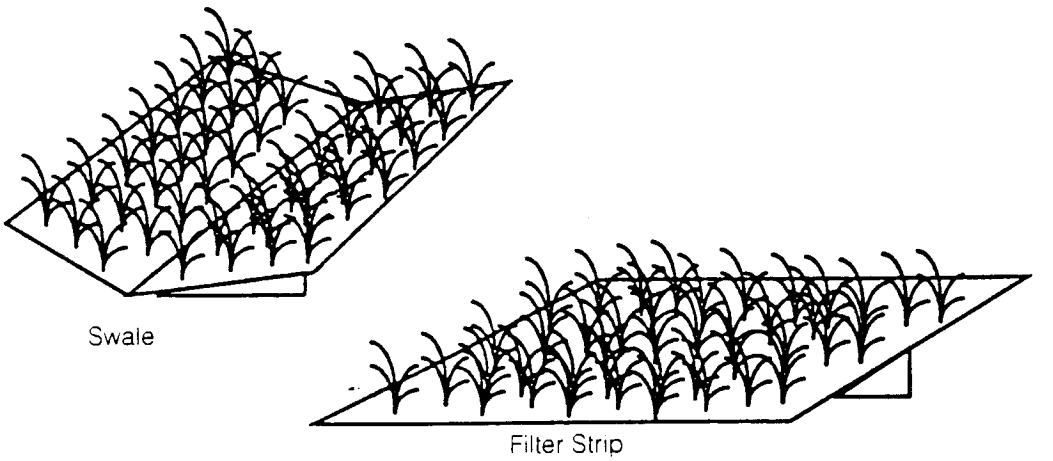


Figure 12.4—Swales and filter strips as controls: efficiency—moderate; function—slow runoff rate, some filtering and infiltration; maintenance-intensive (mowing); sidesloeps of swales must be kept flat; use in combination with other controls.

A filter strip is simply a strip of land across which stormwater from a street, parking lot, rooftop, etc., flows before entering adjacent receiving waters. A summary of the characteristics of swales and filter strips is presented in Figure 12.4.

1. Design Considerations

For small storms, both swales and filter strips remove pollutants from stormwater by slowing the water and settling or filtering out solids as the water travels over the grassed area, and by allowing infiltration into the underlying soil. In general, the higher the flow rate, the lower the efficiency. Thus, low velocity and shallow depth are key design criteria. A swale designed with a low bottom slope and check dams will perform much more efficiently than one without check dams. Raised driveway culverts are also very effective as check dams. For maximum efficiency of pollutant removal during small storms, a trapezoidal swale with as large bottom width as can be fitted into the site plan is desirable, since this will maximize the amount of runoff in contact with the vegetation and soil.

Design flows are calculated using equations for open channel flow (and small, frequent runoff events) with a roughness coefficient suitably adjusted for the grass or vegetation on the channel cross section. If the soil is sufficiently permeable that infiltration through the bottom is significant, this may be taken into account in the channel design. Avellaneda (1985) and Wanielista et al. (1986) provide some guidance in this area. For filter strips, there are two primary design considerations. The first is to minimize the grade in the direction of the flow. One effective technique for this is terracing. The second is to make certain that the water flowing across the strip is introduced at the upstream

side of the strip in such a way that it flows across the strip in sheet flow and does not channelize.

2. *Maintenance*

Maintenance of swales and filter strips is an important consideration, for aesthetic reasons and for hydraulic efficiency. In the case of the swale, care must be taken to insure that flows through a swale used for drainage purposes during large storms are not impeded by an overgrowth of vegetation. To prevent this, the vegetation planted in the channel should be suitable for mowing, and the channel designed so that mowing machines can be easily and efficiently operated along the swale. The swale should be mowed on a regular basis. For filter strips that are not part of the drainageway during large storms, maintenance is purely an aesthetic matter. These strips can be planted in grass and mowed, or natural vegetation can be used. Any ground cover, however, must be sufficiently dense to keep the overland flow from channelizing and eroding rivulets through the filter strip.

C. Porous Pavement and Parking Blocks

Porous pavement has excellent potential for use on streets and in parking areas (Niemczynowicz 1990). When properly designed and carefully installed and maintained, porous pavement can have load-bearing strength and longevity similar to conventional pavement. In addition, porous pavement can help to reduce the amount of land needed for stormwater management to preserve the natural water balance at a given site, and to provide a safer driving surface that offers better skid resistance and reduced hydroplaning.

However, porous pavement is only feasible on sites with permeable soils, fairly flat slopes, and relatively deep water-table and bedrock levels. In addition, batching and placement of the material require special expertise to avoid clogging, which is the principal concern associated with porous pavement. The risk of clogging is high, and, once it has occurred, it is difficult and costly to correct. The chief means of preventing the problem is to keep sediment off the underlying soil before construction and off the pavement during and after construction.

Porous pavement is being used fairly extensively as a viable alternative in Florida, and a design manual has recently become available (Florida Concrete and Products Association 1989). However, where winter conditions are severe, additional consideration should be given to the structural integrity of porous pavement under freeze-thaw conditions. Detailed design information on porous asphalt is available in Schueler (1987) and Maryland Water Resources Administration (1984).

Another very effective site-control device is parking blocks (Pratt 1990). These are hollow concrete blocks similar to but smaller than those used in construction. They are an excellent site control that is unfortunately rarely used. In parking lots for retail stores, sports arenas, civic theaters, and the like, where more than half of the parking area

is used less than 20% of the time, the use of parking blocks in the less-used portions of such lots give them a much more attractive appearance and will considerably reduce runoff quantity, flow rates, and pollution from these areas.

Figure 12.5a shows typical blocks, which are placed in rows with soil surrounding each one and planted with vegetation. Runoff quantity reduction occurs as infiltration takes place in the planted areas. Moreover, the greater flow resistance of the grassed areas retards the runoff rate, especially during small storms. Most importantly, the quality of the runoff is greatly enhanced over that from a normal parking lot because the pollutants, being restrained by the vegetation matrix, are more difficult to wash off.

In designing a parking block area, the block manufacturer should be consulted to determine the most suitable sub-base to use. Also, it is suggested that only the actual parking spaces be paved with the blocks, since they do not hold up well under traffic. Traffic lanes through the lot should be paved in the normal fashion. A typical layout is illustrated in Figure 12.5b.

Finally, it should be noted that parking blocks are best suited for use where the rainfall is frequent enough to keep the planted grass alive. In arid areas, this control should not be used unless provision for watering is made. When the grassed areas are not well-maintained and the grass has been allowed to die, soil erosion inevitably occurs and the parking lot can be muddy.

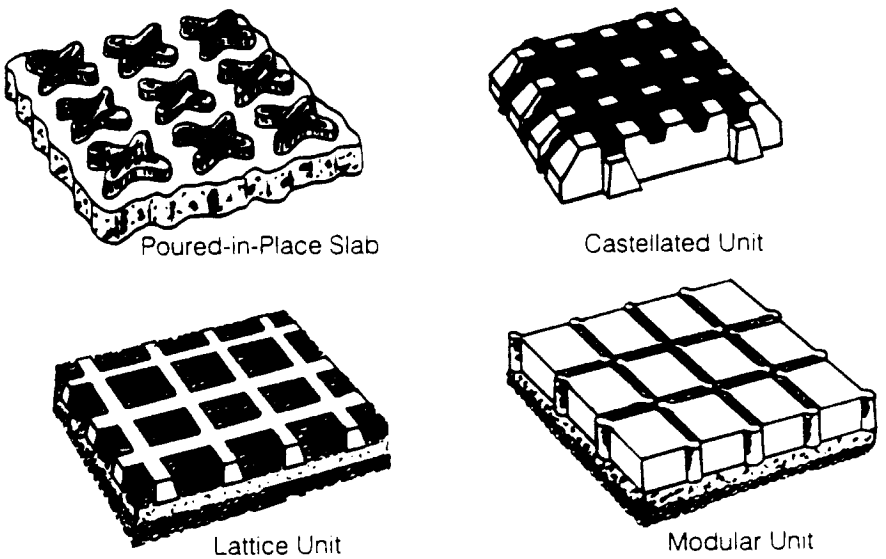
D. Infiltration Devices

Infiltration devices are those stormwater quality control measures that completely capture runoff from the water quality design storm and allow it to infiltrate into the ground. They are the most effective stormwater quality control device that can be implemented, but they can only be used in situations where the captured volume of water can infiltrate into the ground before the next storm, and where they will not cause structural or groundwater pollution problems (particularly if groundwater is used as a potable water source).

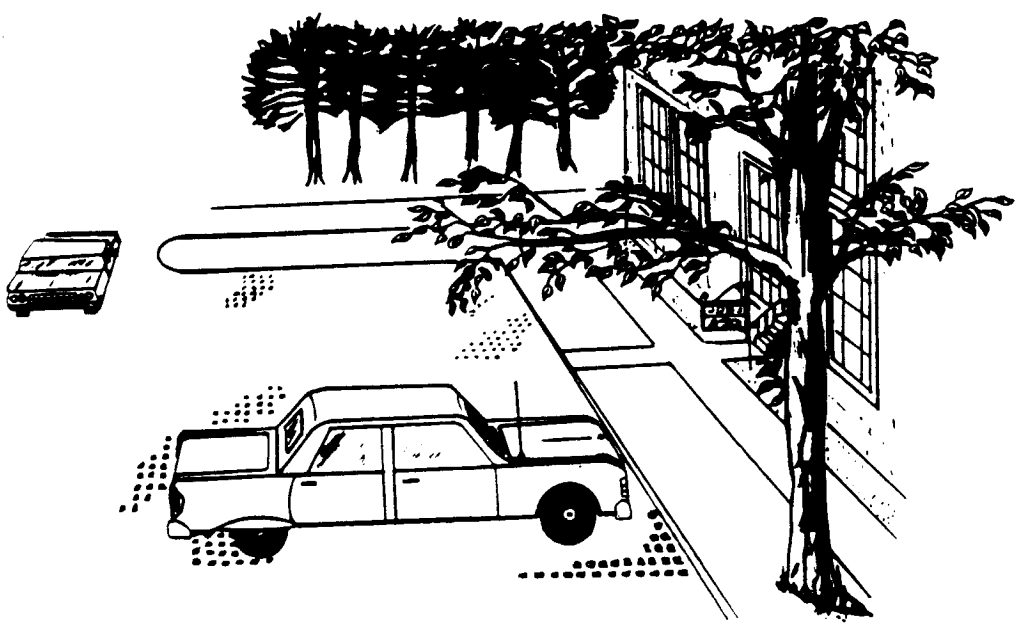
Infiltration devices can be either above-ground infiltration basins or buried infiltration trenches. Among their advantages are that they can help to minimize alterations to the natural water balance of a site, can be integrated into a site's landscaped and open areas, and, if carefully designed, can serve larger developments. Disadvantages can include a fairly high rate of failure due to unsuitable soils, the need for frequent maintenance, and possible nuisance factors, such as odors, mosquitos, or soggy ground. The general performance characteristics of these two types of infiltration device are shown in Figure 12.6.

1. Infiltration Basins

Infiltration basins temporarily store stormwater until it infiltrates the surrounding ground through the bottom and sides of the basin. Such



(a)



(b)

Figure 12.5—(a) Types of grid and modular pavements and (b) typical parking lot layout using grid or modular pavements (Fla. Concrete Prod. Assoc., 1989).

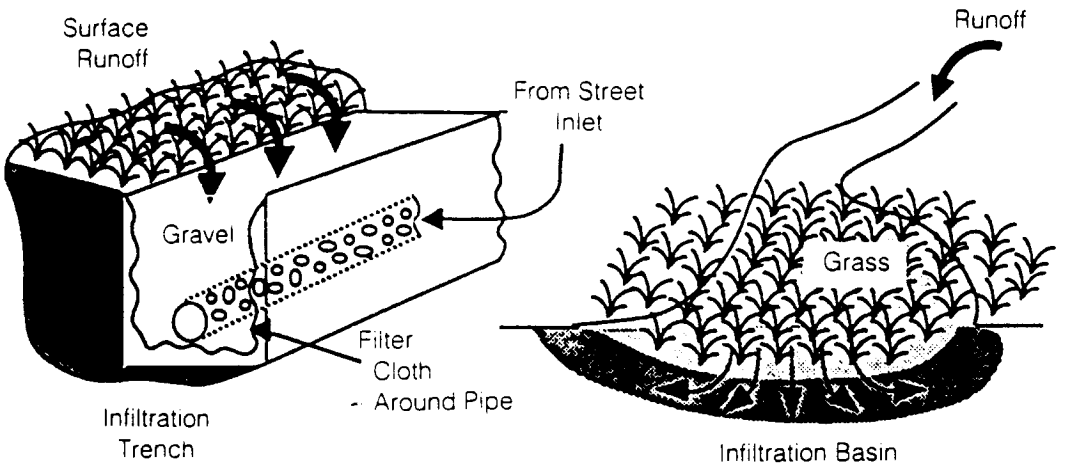


Figure 12.6—Infiltration devices: efficiency—good to excellent (small depressional infiltration basins are excellent onsite controls); function—infiltrates runoff to groundwater, soil filters pollutants; maintenance-intensive (mowing, upstream erosion control); nonfunctional if plugged; soil must be suitably permeable and wet season water table well below bottom; design of underdrained infiltration basins is more difficult and uncertain.

basins are made by constructing an embankment or by excavating in or down to relatively permeable soils. They also can be a natural depression within an open area or a recreational area such as a soccer field. Infiltration basins generally serve areas ranging in size from a front yard to 50 acres.

Infiltration basins can be constructed on-line or off-line with respect to the normal drainage path. When a basin is located on-line, it is designed to capture the water quality design storm entirely. When a larger storm occurs, it overflows the basin, which then serves as a detention pond for those larger events. Experience in Florida indicates that these on-line basins are not as efficient in the removal of stormwater as off-line infiltration basins.

Off-line infiltration basins are designed to divert the more polluted first flush of stormwater out of the normal path and hold it for later water quality treatment. They also help to reduce stormwater volume and to recharge the groundwater, and should be used wherever site conditions allow. When the basin reaches capacity, the flow path for any additional stormwater returns to normal. The diverted first flush is not discharged to surface water but is stored until it is gradually removed by infiltration, evaporation, and evapo-transpiration.

Off-line infiltration areas can often be easily incorporated into the landscaped/open areas of a site. These can include natural or excavated grassed depressions, recreational areas, and even landscape islands in parking lots. However, if site conditions prevent the exclusive use of

infiltration, then small off-line retention areas scattered over the development or site should be used as pre-treatment devices for the runoff prior to subjecting it to the primary runoff treatment device for the development. This is especially necessary if detention lakes are the primary component of the stormwater system and the lakes are intended to serve as an aesthetic focal point of the development (see Section VI).

Off-line infiltration devices are designed to store a selected volume of stormwater for a specified period of time with a predetermined infiltration rate. The success of such practices will depend on the careful evaluation of a potential site for suitability. Special attention must be given to such aspects as soils, bedrock, setbacks, vegetation, and maintenance, as discussed in the following paragraphs.

Infiltration basins must be located in soils with hydraulic conductivity rates that will allow the diverted volume to infiltrate within 72 hours or within 24–36 hours for infiltration areas that are planted with grasses. The seasonal high-water table should be at least three feet beneath the bottom of the infiltration area to assure that stormwater pollutants are removed by the vegetation, soil, and microbes before coming into contact with the groundwater.

Bedrock also should be at least four feet beneath the bottom of the infiltration area. In those parts of the country where limestone is at or near the land surface, the potential for groundwater contamination is quite high, especially in Karst-sensitive areas, where sinkhole formation is common. Similar concerns exist in mountainous areas where thin soil mantles overlay fractured bedrock aquifers that serve as water supplies.

Site selection and planning also must address several other important issues. For example, infiltration basins should not be located on areas with slopes greater than 20%, to minimize the chance of downstream water seepage from the subgrade. Care should be taken to avoid areas with expansive soils, or soils subject to substantial consolidation or settlement upon the introduction of water. In addition, infiltration areas should be set back from water-supply wells or septic systems (distance will vary with site conditions), and at least 10 feet down gradient from any building foundations. Local groundwater quality standards, if any, must be considered, and land used for such purposes as gasoline stations, or industrial lands susceptible to spills of potentially hazardous materials should not use infiltration basins at all unless precautions are taken to prevent such materials or spills from mixing with stormwater.

Once areas with suitable characteristics have been selected for infiltration, the sites should be well marked during surveying and protected during construction. Heavy equipment, vehicles, and sediment-laden runoff should be kept out of infiltration areas to prevent compaction and loss of infiltration capacity. Public safety and owner liability must also be carefully considered during site planning. Additional discussions of the concerns involved in the design and siting of infiltration basins are addressed by Schueler (1987) and Maryland Water Resources Administration (1986).

The side slopes and bottoms of infiltration areas should be planted with a dense turf of native vegetation immediately after construction. This grass should be water-tolerant. Depending on local hydrology. supplemental irrigation at these areas may be required during dry periods. Similarly, inlet channels into and within the basin should be planted with stabilizing vegetation to prevent inflows from reaching erosive velocities and scouring the bottom. Stabilizing these detention areas through the use of vegetation not only helps to eliminate erosion and scouring but also filters stormwater pollutants, reduces maintenance needs, and even maintains or improves infiltration rates. Finally, a maximum slope of 4:1 ensures that maintenance equipment can easily be brought into the site and moved around within it.

2. *Infiltration Trenches*

In general, site planning considerations for infiltration trenches are the same as for infiltration basins. As with basins, trenches must be very carefully designed, installed, and maintained, because they are very susceptible to clogging, and, once clogged, rehabilitation is a major effort. If properly constructed, with pretreatment practices in place to prevent heavy sediment loading, infiltration trenches can provide stormwater benefits without tremendous maintenance needs. Since trenches are usually "out-of-sight, out-of-mind," getting property owners to maintain them can be difficult. Accordingly, a public commitment for regular inspection of privately owned trenches is essential, as is a legally binding maintenance agreement and owner education regarding the function and maintenance needs of trenches.

Infiltration trenches, which can be located on the surface of the ground or buried beneath the surface, are usually designed to serve areas ranging up to 5–10 acres in size and are especially appropriate in urban areas, where land costs are very high. An infiltration trench generally consists of a long, narrow excavation, ranging from 3–12 feet in depth, backfilled with stone aggregate to allow for the temporary storage of the first-flush stormwater in the voids between the aggregate material (See Figure 12.6). Stored runoff then infiltrates into the surrounding soil, through either the trench bottom or the sides, depending on the elevation of the water table and the soil properties. In addition to the infiltration rate of the subsoil, the hydraulic conductivity of the surrounding filter fabric is crucial and, in fact, can become a limiting factor. The use of fabrics designed as landfill liners should be avoided.

Trench bottoms should be at least four feet above the seasonal high water table level to prevent mounding of groundwater, consequent loss of infiltration capacity, and insect problems.

There are two major types of trenches, surface trenches and underground trenches. The differences between the two involve the amount of stormwater that can be handled and the ease of maintenance. Surface trenches receive sheet-flow runoff directly from adjacent areas after it has been filtered by a grass buffer. They are typically used in residential areas where relatively small loads of sediment and oil can be trapped

in grass filters strips at least 20′ wide. While surface trenches may be more susceptible to sediment accumulation, their accessibility makes them easier to maintain. In addition, they are very appropriate for use in highway medians, parking lots, and narrow landscape areas.

Underground trenches can potentially be used in many development situations, although discretion must be exercised. For instance, while underground trenches can accept runoff from storm sewers, they require installation of special inlets to prevent coarse sediment and oils and greases from clogging the stone reservoir. These inlets should include oil/grease traps, catch basins, and baffles to reduce sediment, leaves, and debris (such measures, however, cannot remove petroleum hydrocarbons normally found in urban runoff). In addition, pretreatment by routing the flow over grassed filter strips or vegetated swales is essential to protect the infiltration trench.

The most commonly used underground trench is an exfiltration system in which the stormwater treatment volume is diverted into an oversized perforated pipe placed within an aggregate envelope. The first flush stormwater is stored in the pipe and exfiltrates out of the holes through the gravel and filter fabric and into the surrounding soil. Because of the lack of accessibility, maintenance of these underground trenches can be very difficult and expensive, especially if they are placed beneath parking areas or pavement.

One other type of infiltration trench is the dry well. This device is used extensively in Maryland to store and infiltrate runoff from rooftops. In a dry well, the downspout from the roof gutter is extended into an underground trench which is constructed at least ten feet away from the building foundation. Again, clogging is a concern, and rooftop gutter screens must be used to trap particles, leaves, and other debris. Additional design information on dry wells is available from the Maryland Water Resources Administration (1986).

Trenches should be inspected frequently within the first few months of operation and regularly thereafter. Such inspections should be done after large storms to check for ponding, with water levels in the observation well recorded over several days to check drawdown. In addition, grass buffer strips should maintain a dense, vigorous growth of vegetation; they should receive regular mowing (with bagging of grass clippings) as needed. Finally, pretreatment devices should be checked periodically and cleaned when the sediment reduces available capacity by more than 10% (Schueler 1987).

Design, construction, and maintenance procedures for infiltration trenches of all types can be found in references such as Schueler (1987), Maryland Water Resources Administration (1986), and Stahre (1988), which should be carefully reviewed when use of these devices is planned.

VI. DETENTION PRACTICES

The site controls described above are the most effective and efficient for reducing the pollutant load in urban runoff. Unfortunately, varia-

tions in soil, water table, and geologic conditions throughout the country preclude the use of infiltration practices in many locations. These locales are characterized by slowly percolating soils, subsurface hardpans that restrict infiltration, flat terrain, controls on groundwater quality, and/or high water tables. In such areas detention, either wet or dry, serves as an alternative or supplementary stormwater quality control device.

Detention basins, in fact, are more widely used for stormwater management than any other type of control. However, their primary application to date has been for drainage control, i.e. peak-flow attenuation, rather than for water quality control. The state of practice in the design of detention basins for water quality management is still in its formative stage. Detention basins designed for peak flow attenuation can be given an effective water quality control function at little added cost (see Chapter 11).

The design of both wet and dry detention basins is considered in Chapter 11. The following two sections contain supplemental information that can be incorporated into the design to maximize the pollutant removal efficiency of both types of detention basins.

A. Dry Detention

Detention basins designed for flood peak reduction only are the most common type of detention basin used around the country. These basins generally are not as effective in removing pollutants as wet basins. This is primarily due to the short residence time. For basins with detention times of less than twelve hours, no more than 10% of stormwater pollutants are removed, and, in fact, there may even be a **negative** pollutant removal rate due to the washout of pollutants captured in earlier small storms (Camp Dresser & McKee 1985; Jellerson 1981). Performance can be improved when dry detention basins are designed to include water quality features, as explained in Chapter 11.

Figure 12.7 presents a schematic of a dry detention pond and summarizes typical performance characteristics. There are several design modifications that can be made to a dry detention basin to enhance overall system performance. For example, the use of swales (as pretreatment conveyances) and landscape infiltration will greatly improve the pollutant removal effectiveness and life of a dry detention system. Another possibility is to incorporate a shallow marsh around the outlet, as is shown in Figure 12.8. It should be noted, however, that the addition of a marsh makes this control device no longer a **dry** detention basin but rather a **transitional** device between dry and wet detention. Further discussion of these and other improvements to significantly enhance the pollutant removal performance of dry detention basins can be found in Schueler (1987).

One of the treatment practices that has been commonly used in the southeastern United States for stormwater treatment is detention with filtration, in which stored stormwater is discharged through a filter. Typical filtration systems have included bottom or side-bank sand or

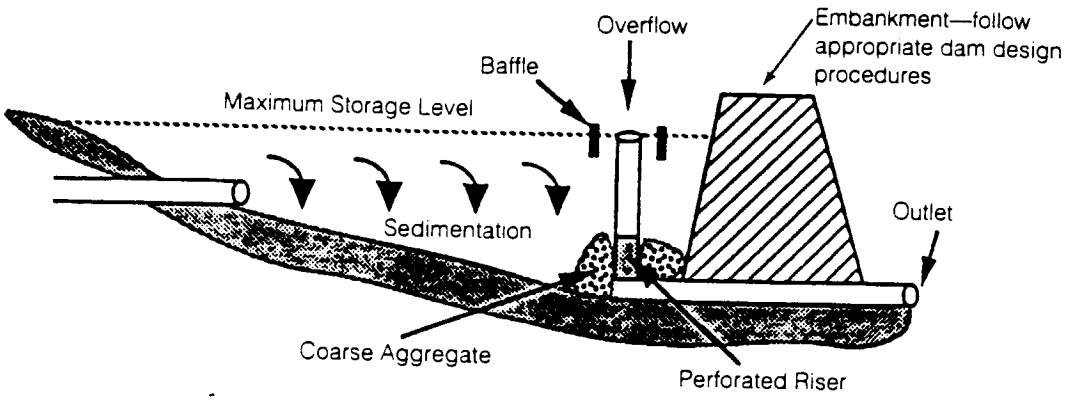


Figure 12.7—Dry detention ponds: efficiency—poor for detention times less than 12 hours and good for detention times greater than 24 hours; function—to allow insoluble pollutants to settle out but soluble pollutants to pass through; maintenance—moderate if properly designed; improper design can make facilities an eyesore and a nuisance in other respects; newer designs are incorporating a shallow marsh around outlet, result—better removal efficiency and reduced mosquito nuisance; regional detention facilities serving 100–200 ac (40–80 ha) can be aesthetically developed and result in lower maintenance costs.

natural soil filters, along with multi-media filters composed of materials such as alum sludge or activated charcoal. Except for the latter two filters, these systems only remove particulate pollutants, which limits their overall benefit.

In addition, difficulties associated with the design, construction, and, most importantly, maintenance of stormwater filters suggests that they should be used only as a last resort. Experience has shown that it is not a question of **if** a filter will clog but **when** (and then who will maintain the filter).

B. Wet Detention Ponds

Figure 12.9 illustrates the basic components of a wet detention system that is used for both flood control and water quality enhancement. The figure also lists the performance characteristics of a typical wet detention basin. Essentially, a wet-detention lake consists of 1) a permanent water pool, 2) an overlying zone in which the design runoff volume temporarily increases the depth of the pool while it is stored and released at the allowed peak discharge rate, and 3) a shallow littoral zone (the biological filter). During storms, runoff replaces treated waters detained within the permanent pool after the previous storm, thus making the permanent water pool volume and the vegetated littoral zone of utmost importance for water quality enhancement. Wet detention ponds are often used in series with swale interconnectors. If properly designed

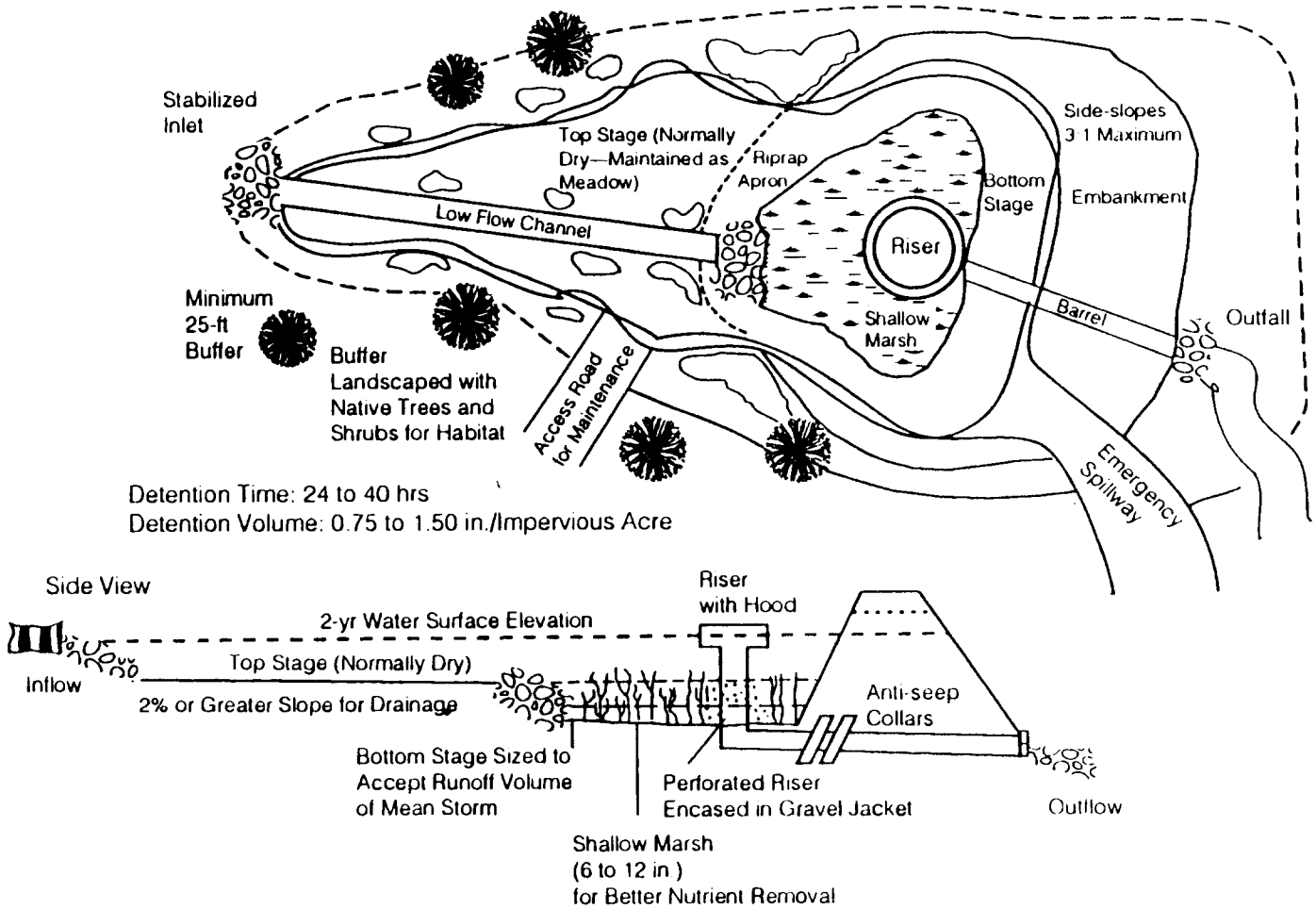


Figure 12.8—Extended detention pond design features (Schueler, 1987).

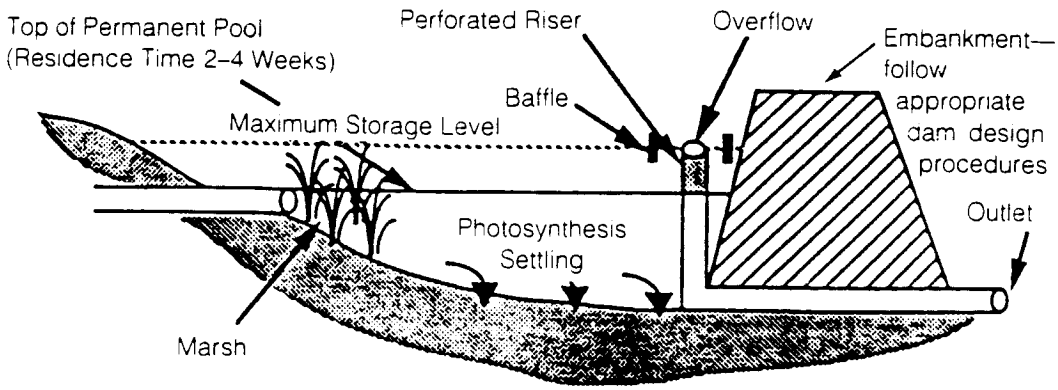


Figure 12.9—Wet detention ponds: efficiency—good to excellent if properly designed, can be poor if bottom goes anoxic; function—removes particulate pollutants by settling, dissolved pollutants biochemically; maintenance—relatively free after first year except for major cleanouts infrequently; aesthetic design can make pond an asset to community, adjacent property actually increases in value; and excellent as a regional facility.

and maintained, they can provide effective flood and water quality protection, as well as an aesthetically pleasing wildlife habitat.

The removal of stormwater pollutants in a wet-detention system is accomplished by a number of physical, chemical, and biological processes. Gravity settling removes particles. Chemical flocculation occurs when heavier sediment particles overtake and coalesce with smaller, lighter particles to form still larger particles. Biological removal of dissolved stormwater pollutants includes uptake by aquatic plants and metabolism by phytoplankton and micro-organisms that inhabit the bottom sediments.

Removal of dissolved pollutants primarily occurs during the relatively long quiescent period between storms. Accordingly, the permanent water pool is especially vital, since it permits treatment between storms, reduces runoff energy, and provides a habitat for aquatic plants and algae (the biological filter that removes dissolved nutrients and metals). Aerobic conditions at the bottom of the permanent pool will maximize the uptake of dissolved pollutants (nutrients, metals) by bottom sediments and will minimize release from the sediments into the water (Yousef 1985).

Designing wet detention systems to achieve a specified level of reduction of dissolved pollutants is very difficult. Two approaches have typically been used to develop design criteria for such systems. One method relies upon solids settling theory and assumes that all pollutant removal is due to sedimentation (Driscoll 1983). The other approach views the wet detention system as a lake that achieves a controlled level of eutrophication, thereby accounting for biological, physical, and chemical assimilation of stormwater pollutants in addition to sedimen-

tation (Hartigan 1985). The basis for this second design method is that stormwater is detained within the permanent pool long enough to produce adequate levels of nutrient uptake by algae and aquatic plants, but the hydraulic residence time is not so long as to induce stagnation, thermal stratification, or anaerobic bottom sediments. Specific design criteria for wet-detention systems using this controlled level of eutrophication approach are described by Hartigan (1988).

There are three critical criteria in determining how efficiently the wet-detention type of device will work. The first is the volume of the permanent pool, which should be sufficient to provide 2–4 weeks of detention time so that algae can grow. Second is the depth of the permanent pool, which should be greater than 4–6 feet but less than 10–15 feet, so that the water remains wind-mixed and the bottom sediment stays aerobic. If the bottom sediment turns anaerobic, it will **release** nutrients into the overlying pool, and these nutrients will be washed out in the next rainstorm.

The third criterion involves the presence of a shallow littoral zone, typically concentrated around inflow points and the outflow (If vegetation is planted continuously around the perimeter, experience shows that homeowners tend to remove it). This is one of the very important components of the system, since it is the aquatic plants within this zone that provide much of the biological assimilation of dissolved stormwater pollutants. The littoral zone should cover at least 30% of the pond's surface area and should have a gentle slope (6:1 or shallower) to a depth of two feet below the control elevation. Another benefit of flat side slopes is enhanced public safety, especially for children. Figure 12.10 by Scheuler (1987) shows an example of a wet-detention pond sized for 2- to 4-week residence time.

A detention system with a permanent water pool must receive and keep enough water to maintain the pool. In some arid parts of the United States, this is not possible. The volume of stormwater that will be detained can be determined from the pond's drainage area, the amount of imperviousness, soil types, and water table elevation. Wet ponds should not be considered in areas where the underlying soil infiltration rate will not allow water to remain in the pond on a permanent basis or where the rainfall is not sufficient to sustain the necessary volume in the pool.

The use of several detention lakes in series or the separation of a pond into multiple cells will enhance pollutant removal and lessen maintenance tasks. In fact, the configuration of the components of a wet-detention system is limited only by the designer's imagination and site constraints. Figure 12.11 illustrates a multi-cell design in which the relatively small deep pool serves as the primary sedimentation area (which can easily be dredged) and the larger shallower pool provides for flood protection and pollutant attenuation. The vegetated littoral area is concentrated in two spots—a shallow ledge between the two pools and near the outlet structure.

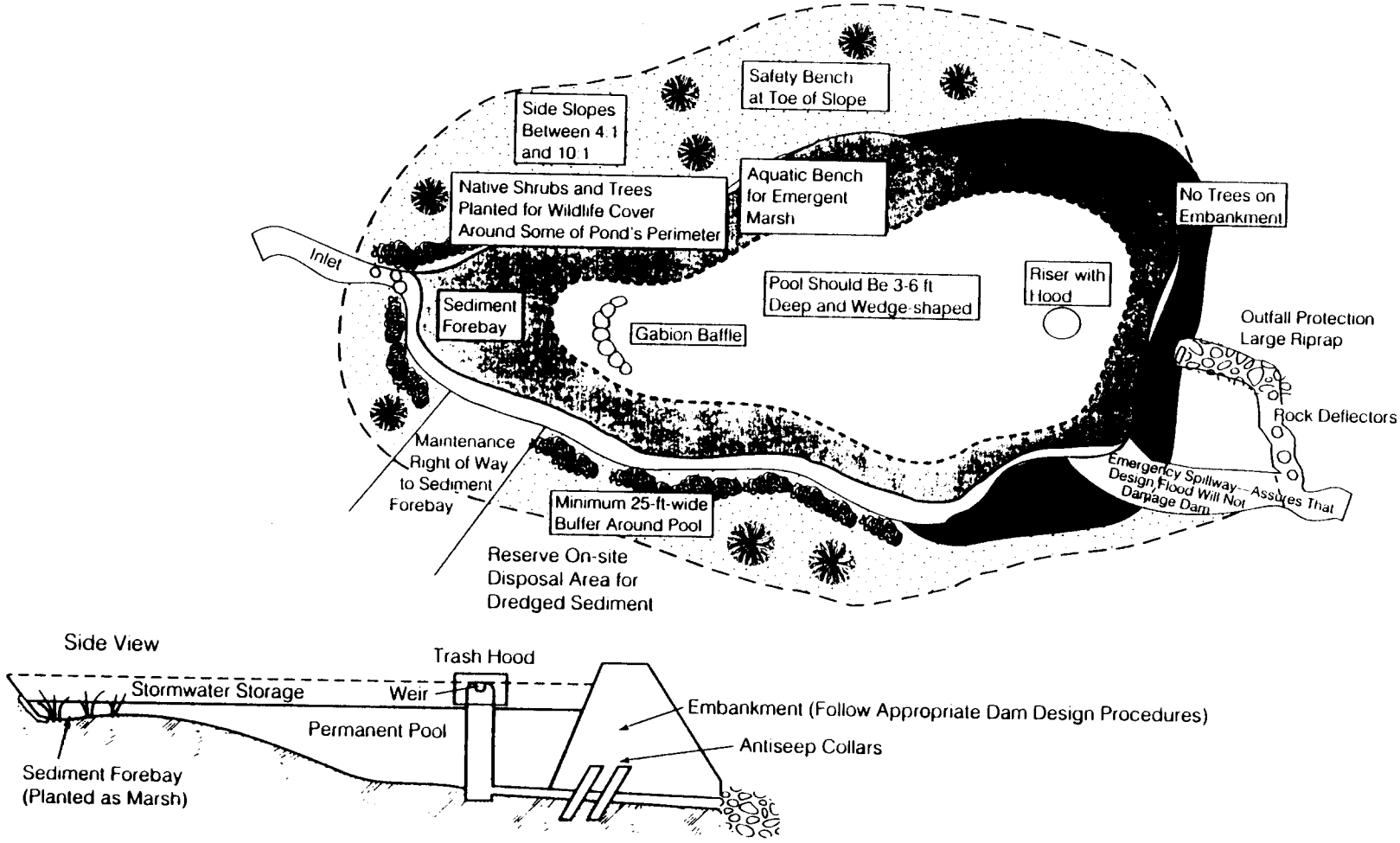


Figure 12.10—Design schematic of a wet pond (ft × 0.304 8 = m) (adapted from Schueler, 1987, with modifications).

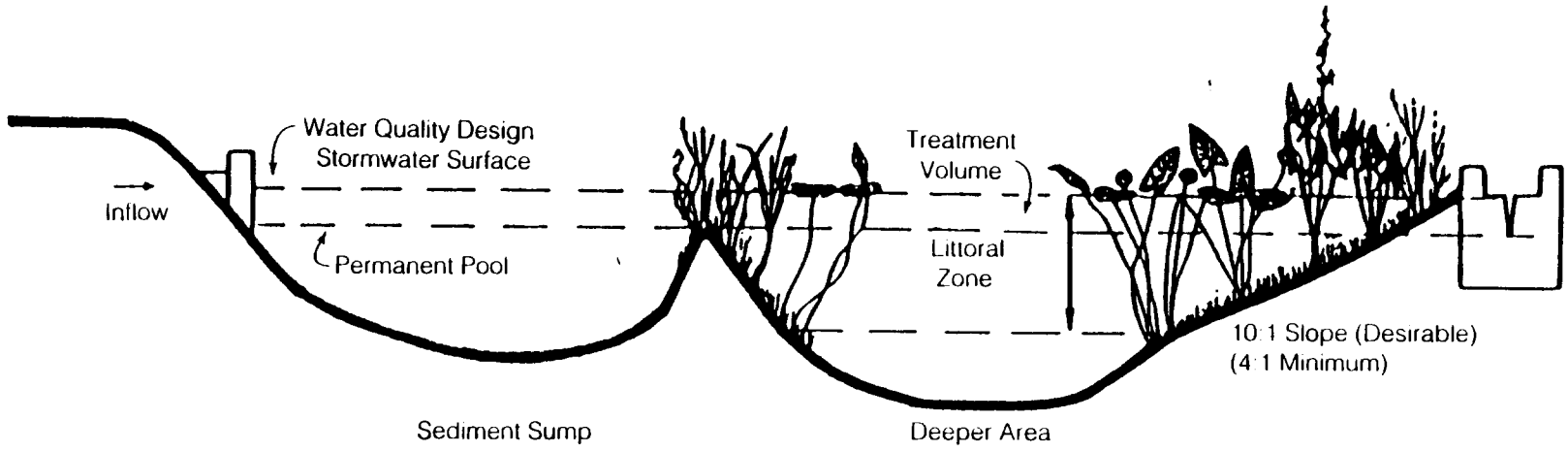


Figure 12.11—Wet detention pond—multi-cell design.

VII. USING WETLANDS FOR STORMWATER QUALITY ENHANCEMENT

Wetlands can provide water quality enhancement through sedimentation, filtration, absorption, and biological processes, as well as natural flood protection. Further, the incorporation of wetlands into a comprehensive stormwater management system achieves several additional objectives, including reduced operation/ maintenance and wetland preservation and revitalization.

However, the use of wetlands for stormwater management should not be considered a panacea, nor should it be considered applicable in much of the United States. While much research has been completed on the ability of wetlands to remove wastewater pollutants (USEPA 1985), many questions still remain. For example, how long can a wetland continue to remove stormwater pollutants effectively? What type of maintenance (harvesting, sediment banking, or other) should be planned for a given wetland area? How frequently should an area be scheduled for this maintenance? These and other such questions must be addressed in order to design the most effective wetland stormwater management systems. In addition, for both fresh and saltwater wetlands there are laws and regulations (see Chapter 2) severely limiting such possibilities.

Given the many unknowns still associated with this treatment practice, only relatively isolated wetlands or constructed wetlands should be used for stormwater management. Wetlands that have been previously ditched and drained can be revitalized by blocking the drains and incorporating them into the stormwater system. Wetlands intermittently connected to other waters through flows that result when ground water rises to ground level should also be considered for stormwater management.

In designing a stormwater management system that incorporates a wetland, care should be taken to minimize changes to the hydro-period (the time that water remains at a particular level in the wetland—which determines the form, function, and nature of the wetland) caused by the addition of stormwater. Finally, pretreatment practices are needed to reduce oil, grease, and sediment levels to protect the wetland. Design guidance in these and other matters relative to constructed wetlands can be found in Maryland Water Resources Administration (1987) and Livingston (1989).

VIII. ALUM TREATMENT OF STORMWATER

Alum (aluminum sulfate) has been used extensively to clarify potable water supplies, to remove phosphorus from wastewater and to inactivate phosphorus in lake sediments. Injection of liquid alum inside storm sewers to treat stormwater is innovative and has potential for

application in reducing the water quality impact of stormwater discharges on lakes.

Alum forms harmless precipitates of $Al(PO_4)$ and $Al(OH_3)$ that combine with phosphorus and heavy metals, causing them to be deposited into the sediments in a stable inactive state. The insoluble precipitates formed between alum and phosphorus in the sediments are exceptionally stable, since they are immune to changes in sediment redox potential and, to a lesser degree, to pH changes.

Among the advantages of injecting alum into stormwater are the excellent reduction of the amounts of most stormwater pollutants (to more than 80%), the relatively low construction and operation costs, the removal of nutrients from the receiving water column, and the reduction of nutrient recycling from bottom sediments.

On the other hand, the disadvantages of using alum include the experimental nature of the technology, the need for careful evaluation of alum dosage levels, the potential for pH changes in the receiving water, and the possible toxicity of the aluminum compound. However, it is known that if alum injections into a particular body of water do affect the benthic organisms residing in bottom sediments, those effects are not long-lasting. For more information on this type of treatment, see Harper, et al. (1986).

IX. IMPLEMENTING STORMWATER QUALITY CONTROLS

Integrating urban runoff quality controls into a drainage plan requires creativity. During the site-planning phase, the designer/planner must constantly have in mind the four objectives of the stormwater management. These are:

(a) Surface drainage.
(b) Flood control.
(c) Erosion and sediment control.
(d) Control of pollutants in the runoff.

The approach to runoff water quality control is to minimize the adverse impacts of stormwater through a coordinated system of source controls and site controls. Source controls emphasize the prevention and reduction of nonpoint pollution by eliminating the opportunity for pollutants on the land surface to be entrained into surface runoff. They are fundamental to the various management methods. After that, a coordinated set of site controls, detention devices, and other practices is necessary to achieve the desired level of runoff quality control. In order for this second step to be both aesthetically acceptable and cost-effective, the stormwater management system must be an integral part of the site-planning process for every project.

Each site will contain natural attributes that should influence the type and configuration of the stormwater system. For example, sandy soils

imply the possibility to integrate infiltration practices such as retention areas into the development's open space and landscaping, while natural low areas and isolated wetlands may offer opportunities for detention/wetland treatment. The variety of features contained on a site will often suggest which particular combination of runoff quality controls can best be integrated into an effective system.

A stormwater management system can be viewed as a treatment train in which the individual treatment practices are the cars. The first car is source controls, followed by site controls, such as minimizing DCIA, filter strips, and infiltration, that capture and remove pollutants near their point of entrainment in the runoff. Grassed swales, detention ponds, wetlands, and other devices add to the train. The more controls incorporated into the system, the better the performance of the train.

It should be noted that the preceding discussion applies primarily to the establishment of new regulatory criteria, and to governmental programs such as best management practices and nonpoint source control programs. The engineer charged with developing designs for a particular client must of course conform to regulatory criteria, including any requirements resulting from regional planning of stormwater management systems, requirements in the interest of flood peak reduction and erosion control, and any specific environmental constraints. He must include design provisions to satisfy these constraints, and also any additional water quality control provisions (such as those in this chapter) that can be included at nominal cost. He can thus satisfy governmental requirements, protect the interests of his client, and produce the most environmentally favorable design possible.

If the design is developed with the following concepts in mind, a good water quality management system will result.

(a) Design runoff quality controls to capture small storms.
(b) Design to maximize sediment removal, and removal of other pollutants.
(c) The most effective method for reducing urban runoff pollution is to minimize directly connected impervious area (DCIA).
(d) Infiltration devices are most efficient but most difficult to maintain (and can only be used where groundwater pollution is not a problem).
(e) Dry detention is easiest to design and operate, and efficiency is satisfactory if properly designed.
(f) Wet detention is more difficult to design and maintain but more efficient than dry detention, and often more aesthetically desirable.

It can be extremely difficult (and expensive) to insert detention facilities into areas that are already intensively developed—the NIMBY (not in my back yard) syndrome is strong. It is possible, however, with thoughtful planning and careful design, to integrate cost-effective runoff quality controls into urban development plans to achieve the required level of pollutant reduction with no negative impact on aesthetics. When all is said and done, the incorporation of runoff quality controls into urban landscape design is more an art than a science. If properly done, the aesthetic character of the development is actually enhanced by integration of runoff quality controls into the site plan.

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CHAPTER 13

MATERIALS OF CONSTRUCTION AND MAINTENANCE

I. INTRODUCTION

A. General

Most man-made drainage works throughout recorded history have served the field of agriculture. The age of the Pharaohs was made possible by the Egyptians' ingenious use of the Nile floods, and man has continually sought to understand and to utilize nature's way of draining the earth of storm waters. Numerous examples, however, have shown that nature will return storm drainage patterns to their natural state unless the man-made improvements are maintained continuously. The concept of forever is not a possibility available to the engineer. All drainage structures have a usable life, and we must design and utilize materials to handle drainage for specific lifetimes.

B. Environmental Considerations

There are many materials available to the engineer designing a storm water conveyance system. The environment in which a given material will be installed will greatly influence its useful service life. Soil and water Ph, soil resistivity, stream silt load, stream geology, and other environmental factors combine to limit or extend the usable life of the planned storm drainage system.

If possible, the designer should analyze the chemical characteristics of the liquid that is being transported by the system as well as the soil characteristics in which the system is to be installed. These analyses will be helpful in selecting the appropriate material and ensuring a more durable system.

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C. Economic Considerations

The drainage system's initial cost and estimated usable life are most important to the engineer. The concept of life cycle cost analysis for engineering systems was introduced in Chapter 8. Knowing the cost of materials and installation, the cost of money over the projected life cycle of the facility, and the cost of maintaining the facility, a total comparative cost in terms of present dollars can be derived which allows the engineer to compare alternative systems in a clear and objective manner.

Different sections of the country have different needs that strongly influence the type of materials appropriate for a specific project. The cost of materials in various regions of the United States strongly influence the life cycle cost of the facility. The engineer should therefore spend a significant amount of design time and study in unit-cost comparison of available materials, prior to actual specification. Various sources of information are listed in the reference section of this chapter.

D. Materials

The evaluation and selection of materials for sewer construction depend on the anticipated conditions of service. Consider the following factors:

(a) Intended use.
(b) Scour or abrasion conditions.
(c) Installation requirements—pipe characteristics and sensitivities.
(d) Corrosion conditions—chemical and biological factors both within the pipe and in the surrounding soil.
(e) Flow requirements—pipe size, velocity, slope and friction coefficient.
(f) Product characteristics—cross sectional shapes, fitting and connection requirements, laying strength, supplementary protective coating systems.
(g) Cost effectiveness—materials, installation, maintenance, life expectancy.
(h) Physical properties—crush strength for rigid pipe, pipe stiffness or stiffness factor for flexible pipe, soil conditions, pipe loading strength, pipe shear loading strength, pipe flexural strength.
(i) Handling requirements—weight, impact resistance.

No single product will provide optimum capability in every characteristic for all design conditions. Specific application requirements should be evaluated prior to selecting or specifying materials. Many studies have been conducted concerning durability and are available to the designer and/or owner to facilitate design and construction of a relatively maintenance-free storm sewer system. New materials are continually being offered for use in storm drainage construction. The discussion has been limited to the commonly accepted materials currently available today.

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II. MATERIALS FOR OPEN, LINED CHANNELS

Open, lined channels are used extensively throughout the world as water conduits. Their performance depends largely on the suitability of the channel lining. For a comprehensive discussion of channel lining materials and the design criteria for their use, see Chapter 9.

III. CONDUIT MATERIALS

Because of the maintenance costs involved with open channel drainage systems, the drainage design engineer should always investigate the feasibility of an enclosed conduit system in each design situation. This section deals with the materials available for the construction of closed conduits. Details on the structural aspects of design will be found in Chapter 14.

A. Rigid Pipe

Pipe materials in this classification derive a substantial part of their basic earth load carrying capacity from the structural strength inherent in the rigid pipe wall. Commonly specified rigid pipe materials include:

1. Asbestos Cement Pipe (ACP)

Asbestos-cement pipe has been used for both gravity and pressure sanitary sewers but has now become less common. The product, produced from asbestos fiber and cement, is available in nominal diameters from 4–36" and in some areas up to 42 inches. A full range of compatible fittings is available. Jointing is accomplished by compressing elastomeric rings between pipe ends and sleeves or couplings.

ACP manufactured for gravity drain applications is available in seven strength classifications. The class designation represents a minimum crushing strength of the pipe expressed in pounds per linear foot of pipe. Potential advantages of asbestos-cement pipe include:

(a) Long laying lengths (in some situations).
(b) Wide range of strength classifications.
(c) Wide range of fittings available.

Potential disadvantages of asbestos-cement pipe include:

(a) Subject to corrosion where acids are present.
(b) Subject to shear and beam breakage when improperly bedded.
(c) Low beam strength.

Asbestos-cement pipe is specified by pipe diameter and class or strength. The product should be manufactured in accordance with one or more of the following specifications:

(a) "Standard Specification for Asbestos-Cement Non-Pressure Pipe," ASTM C 428.
(b) "Standard Specification for Asbestos-Cement Non-Pressure Small Diameter Sewer Pipe," ASTM C 644.

2. *Cast Iron Pipe (CIP)*

CIP (gray iron) has been used for both gravity and pressure drainage systems, although recently ductile iron pipe has generally been specified in its place. Standards specify the product in nominal diameters from 2–48" inches with a variety of joints. Cast iron fittings and appurtenances are generally available. Product availability is limited, as manufacturers are converting to ductile iron production.

CIP is manufactured in a number of thicknesses, classes, and strengths. A cement mortar lining with an asphaltic seal coating may be specified on the interior of the pipe. An exterior asphaltic coating is also commonly specified. Other linings and coatings may be specified. Potential advantages of cast iron pipe (gray iron) include:

(a) Long laying lengths (in some situations).
(b) High pressure and load bearing capacity.

Potential disadvantages of cast iron pipe (gray iron) include:

(a) Subject to corrosion where acids are present.
(b) Subject to chemical attack in corrosive soils.
(c) Subject to shear and beam breakage when improperly bedded.
(d) High weight per length ratio.
(e) Product availability.

CIP is specified by nominal diameter, class, lining, and type of joint. CIP is manufactured in accordance with one or more of the following standard specifications:

(a) "Cast Iron Pipe Centrifugally Cast in Metal Molds, for Water or Other Liquids," ANSI A 21.6 (AWWA C 106).
(b) "Gray-Iron and Ductile Iron Fittings, 2 through 48-inch, for Water and Other Liquids," ANSI/AWWA C 110.
(c) "Polyethylene Encasement for Gray and Ductile Iron Piping for Water and Other Liquids," ANSI/AWWA C-105/A 21.5.
(d) "Flanged Cast-Iron and Ductile-Iron Pipe with Threaded Flanges," ANSI A 21.15 (AWWA C 115).
(e) "Cement Mortar Lining for Cast-Iron and Ductile Iron Pipe and Fittings for Water," ANSI A 21.4 (AWWA C-104).

Additional information relative to the selection and design of CIP may be obtained from the Ductile Iron Pipe Research Association (1984).

3. *Concrete Pipe*

Reinforced and non-reinforced concrete pipe are used for gravity storm drainage. Reinforced concrete pressure pipe and prestressed con-

crete pipes are used for pressure as well as gravity drains. Non-reinforced concrete pipe is available in nominal diameters from 4–36". Reinforced concrete pipe is available in nominal diameters from 12–200". Pressure pipe is available in diameters from 12–120". Concrete fittings and appurtenances such as wyes, tees, and manhole sections are generally available. A number of jointing methods are available depending on the tightness required and the operating pressure, and various linings and coatings are available. A number of mechanical processes are used in the manufacture of concrete pipe, including centrifugation, vibration, and packing and tamping. Gravity and pressure concrete pipe may be manufactured to any reasonable strength requirement by varying the wall thickness, concrete strength, quantity and configuration of reinforcing steel or prestressing elements. Potential advantages of concrete pipe include:

(a) Wide range of structural and pressure strengths.
(b) Wide range of nominal diameters.
(c) Wide range of standard lengths (generally 4–24').
(d) Moderately low friction losses.
(e) Resistance to galvanic corrosion and abrasion.

Potential disadvantages of concrete pipe include:

(a) High weight.
(b) Subject to corrosion where acids are present.

Concrete pipe is normally specified by nominal diameter, class or D-load strength and type of joint. The product should be manufactured in accordance with one or more of the following standard specifications.

(a) "Concrete Sewer, Storm Drain, and Culvert Pipe," ANSI/ASTM C 14.
(b) "Reinforced Concrete Culvert, Storm Drain, and Sewer Pipe," ANSI/ASTM C 76.
(c) "Reinforced Concrete Arch Culvert, Storm Drain, and Sewer Pipe," ANSI/ASTM C 655.
(d) "Reinforced Concrete Elliptical Culvert, Storm Drain, and Sewer Pipe," ANSI/ASTM C 507.
(e) "Reinforced Concrete Box Culverts," ANSI/ASTM C250, C789M, and C789.
(f) "Reinforced Concrete Low-Head Pressure Pipe," ANSI/ASTM C 361.
(g) "Joints for Circular Concrete Sewer and Culvert Pipe, Using Rubber Gaskets," ANSI/ASTM C 443.
(h) "External Sealing Bands for Non-Circular Concrete Sewer, Storm Drain, and Culvert Pipe," ANSI/ASTM C 877.

Additional information relative to the selection and design of concrete pipe may be obtained from the American Concrete Pipe Association (1970, 1988).

4. *Vitrified Clay Pipe (VCP)*

VCP, manufactured from clay and shales, is used for gravity storm drainage. The pipe is vitrified at a temperature at which the clay mineral particles become fused. The product is available in diameters from 3–36″ and in some areas up to 42″. Clay fittings are available to meet most requirements, with special fittings manufactured upon request. A number of jointing methods are available.

VCP is manufactured in standard and extra-strength classifications, although in some areas the manufacture of standard-strength pipe is not common in sizes 12″ and smaller. The strength of vitrified clay pipe varies with the diameter and strength classification. The pipe is manufactured in lengths up to 10′. Potential advantages of vitrified clay pipe include:

(a) High resistance to chemical corrosion.
(b) High resistance to abrasion.
(c) Wide range of fittings available.
(d) Low friction losses.

Potential disadvantages of vitrified clay pipe include:

(a) Limited range of sizes available.
(b) High weight.
(c) Subject to shear and beam breakage when improperly bedded.
(d) Low beam strength.

VCP is specified by nominal pipe diameter, strength, and type of joint. The product should be manufactured in accordance with one or more of the following standard specifications:

(a) "Standard Specification for Vitrified Clay Pipe, Extra Strength, Standard Strength and Perforated," ANSI/ASTM C 700.
(b) "Compression Joints for Vitrified Clay Pipe and Fittings," ASTM C 425.
(c) "Pipe, Clay, Sewer," Federal Specification SS-P361d, Standard Methods of Testing Vitrified Clay Pipe, ANSI/ASTM 301.
(d) Crushing Strength for Pipe and Fittings for Perforated VCP in Accordance with NCPI ER4–67.

Additional information relative to the selection and design of vitrified clay pipe may be obtained from the National Clay Pipe Institute (1978).

B. Flexible Pipe

Storm drainage pipe materials in this classification derive their load carrying capacity from a combination of the inherent strength of the pipe and of the interaction of the flexible pipe and the embedment soils (affected by the deflection of the pipe under load). Commonly specified flexible pipe materials are discussed below:

1. *Ductile Iron Pipe (DIP)*

DIP is used for both gravity and pressure drains. DIP is manufactured by adding cerium or magnesium to cast (gray) iron just prior to the casting process. The product is available in nominal diameters from 3–54" and in lengths to 20'. Cast iron (gray iron) or ductile iron fittings are used with ductile iron pipe. Various jointing methods for the product are available.

Applications of DIP generally involve one or more of the following conditions, high impact and/or loading, minimal cover, and long service life with minimal maintenance.

DIP is manufactured in various thicknesses, classes and strengths. Pipe linings such as cement mortar lining with asphaltic coating, coal tar epoxies, epoxies, and polyethylene may be specified. An exterior asphaltic coating and polyethylene exterior wrapping are also commonly specified. Potential advantages of DIP include:

(a) Long laying lengths (in some situations).
(b) High impact strength.
(c) High pressure and load bearing capacity.
(d) High beam strength.
(e) Low friction losses.

Potential disadvantages of DIP include:

(a) Subject to corrosion where acids are present.
(b) Subject to chemical attack in corrosive soils.
(c) High weight.

DIP is specified by nominal diameter, class, lining, and type of joint. DIP should be manufactured in accordance with one or more of the following standards specifications:

(a) "Polyethylene Encasement for Gray and Ductile Cast-Iron Piping for Water and Other Liquids," ANSI A 21.5 (AWWA C 105).
(b) "Ductile Iron Gravity Sewer Pipe," ASTM A 746.
(c) "Gray-Iron and Ductile Iron Fittings, 3 inch through 48 inch, for Water and Other Liquids," ANSI/AWWA C 110.
(d) "Cement Mortar Lining for Cast-Iron and Ductile-Iron Pipe and Fittings for Water," ANSI A 21.4 (AWWA C 104).

Additional information relative to the selection and design of DIP may be obtained from the Ductile Iron Pipe Research Association (1984).

2. *Fabricated Steel Pipe*

Corrugated Steel Pipe, Arches, and Pipe Arches—galvanized corrugated steel is fabricated in a variety of conduit shapes with a choice of additional protective coatings when deemed necessary. Available sizes and shapes include: circular, in diameters from 12–144"; pipe arches manufactured from circular pipe from 15–120" in diameter; struc-

tural plate structures of 60–312″ in diameter; structural plate arches from 6–25′ in span with concrete base; and circular, arch, or horseshoe in shape. Strengths to meet a variety of design loads may be obtained by specifying from a range of gages and joint types.

Pipe sections are furnished in standard lengths of 20, 30, or 40′ in multiples of 2 or 4′. Shorter or longer lengths can be provided by the fabricator, as lock seam or welded seam pipe is manufactured from a continuous coil and can be cut to any length. The sections are joined by coupling bands which may be single piece, two piece, or of an internal expanding type used in lining work. Large sizes (structural plate conduits) are field-bolted. Corrugated pipe is also available in helical corrugations with improved flow characteristics.

Appurtenances include tees, wyes, elbows, and manholes fabricated from the same corrugated material as well as end sections. Corrugated pipe may be designed specially for jacking purposes. Advantages of corrugated steel pipe, arches, and pipe arches include:

(a) Light weight.
(b) Long laying lengths (in some situations).
(c) Flexibility.
(d) Usefulness as a lining for the repair of existing structures.
(e) Wide range of coatings available.

The principal disadvantages are:

(a) Relatively poor hydraulic coefficient unless fully lined with bituminous materials.
(b) Subject to corrosion in aggressive environments.
(c) The requirement for structurally satisfactory horizontal support of the spring-line of the upper semi-circular arch of the pipe or culvert (special attention must be given to proper bedding).

Bituminous linings, which cover the crests of the corrugations and form a smooth surface to improve flow characteristics, sometimes are used to improve hydraulic performance (the invert may also be paved), and to impart corrosion resistance. An inherent problem with coated corrugated steel pipe is the achievement of an effective bond between the coating and pipe. Durability is increased with coating, but the coated material should not be expected to remain intact for the full life of the pipe. Smooth-coated corrugated steel pipe should not be used in outfall situations where the materials are subject to freezing and thawing conditions. External corrosion protection may be necessary depending on soil conditions. Bituminous coatings are flammable and may be damaged or destroyed by petroleum wastes or solvents. Care should also be taken in handling coated pipe during installation to prevent damage to the coating. Continuous adequate lateral support is critical to obtain structural stability of the pipe.

Corrugated steel pipe, arches, or pipe arches are specified by size (nominal diameter, span and rise, or arc length), shape (full circle,

elliptical, arch, or segmental plate arch), gage of metal (depending on strength requirements), assembly of sections with bands or bolts, coatings or linings, and couplings (width of single piece and two-piece).

Additional information relative to the selection and design of steel pipe may be obtained from the American Iron and Steel Institute (1971), and a number of books published by the National Corrugated Steel Pipe Association.

3. *Corrugated Aluminum Pipe*

Corrugated aluminum pipe, arches, and box culverts are available in many sizes and shapes. Circular pipe is available from 6–180 in. Arches can be specified in a variety of sizes up to 14 foot rise by 30 foot span. Box culverts are available up to 10 feet-2 inch rise by 25 feet-5 inch span. The strength of pipe is specified by a range of gages, joint types (welded, mechanical coupling, bolted), and method of bedding or backfilling. Pipe sections can be furnished in lengths up to 40 ft. and are easy to handle due to their light weight. Appurtenances include tees, wyes, elbows, manholes and end sections. Large size arches and box culverts are generally field bolted. Smaller sizes can be cut and welded in the field and are extremely versatile for on-site fabrication. Advantages of corrugated aluminum pipe, arches and box culverts include:

(a) Light weight.
(b) Long laying lengths.
(c) Ease of field connection and fabrication.
(d) Flexibility.
(e) Durability and resistance to corrosive environments such as saltwater.

Disadvantages include:

(a) Relatively poor hydraulic coefficient unless lined with bituminous material.
(b) The requirement for structurally satisfactory horizontal support (see corrugated steel pipe).

The aluminum pipe should conform to one or more of the following standard specifications:

(a) Standard Specification for Corrugated Aluminum Alloy Culverts and Underdrains, AASHTO Designation M-196.
(b) Standard Specification for Aluminum Structural Plate for Pipe, Pipe Arches, and Arches, AASHTO, Designation M-219.
(c) Standard Specification—For Clad Aluminum Alloy Sheets for Culverts and Underdrains, AASHTO, Designation M-197.
(d) Federal Specification—Pipe, Corrugated (Aluminum Alloy) WW-P-402.

Additional information relative to selection and design of corrugated aluminum pipe may be obtained from the Aluminum Association.

4. *Thermoplastic Pipe*

Thermoplastic materials include a broad variety of plastics that can be repeatedly softened by heating and hardened by cooling through a temperature range characteristic for each specific plastic. Generally, thermoplastic materials used in sanitary sewers and drainage are limited to acrylonitrile-butadiene-styrene (ABS), polyethylene (PE), and polyvinyl chloride (PVC).

Acrylonitrile-Butadiene-Styrene (ABS) Pipe ABS pipe is used for both gravity and pressure drains. Non-pressure rated ABS sewer pipe is available in nominal diameters from 3–12″ and in lengths up to 35′. A variety of ABS fittings and several jointing systems are available.

ABS pipe is manufactured by extrusion of ABS plastic material, and is available in three dimension ratio (DR) classifications (23.5, 35, and 42) depending on nominal diameter selected. The classifications relate to three pipe stiffness values, PS 150, 45 and 20 psi, respectively. The DR is the ratio of the average outside diameter to the minimum wall thickness of the pipe. Potential advantages of ABS pipe include:

(a) Light weight.
(b) Long laying lengths (in some situations).
(c) High impact strength.
(d) Ease in field cutting and tapping.

Potential disadvantages of ABS pipe include:

(a) Limited range of sizes available.
(b) Subject to environmental stress cracking.
(c) Subject to excessive deflection when improperly bedded and haunched.
(d) Subject to attack by certain organic chemicals.
(e) Subject to surface change affected by long-term ultra-violet exposure.

ABS pipe is specified by nominal diameter, dimension ratio, pipe stiffness and type of joint. ABS pipe should be manufactured in accordance with one or more of the following specifications:

(a) "Acrylonitrile-Butadiene-Styrene (ABS) Sewer Pipe and Fittings," ANSI/ASTM D 2751.
(b) "Solvent Cement for Acrylonitrile-Butadiene-Styrene (ABS) Plastic Pipe and Fittings," ANSI/ASTM D 2235.
(c) "Elastomeric Seals (Gaskets) for Jointing Plastic Pipe," ANSI/ASTM F 477.
(d) "Joints for Drain and Sewer Plastic Pipes Using Flexible Elastomeric Seals," ANSI/ASTM D 3212.
(e) "PVC and ABS Injected Solvent Cemented Pipe Joints," ANSI/ASTM F 545.

Acrylonitrile-Butadiene-Styrene (ABS) Composite Pipe ABS composite pipe can be used for gravity drainage pipes. The product is available in nominal diameters from 8–15″ and in lengths from 6.25–12.5′. ABS

fittings are available for the product. The jointing systems available include elastomeric gasket joints and solvent cemented joints.

ABS composite pipe is manufactured by extrusion of ABS plastic material with a series of truss annuli which are filled with filter material such as light-weight portland cement concrete. Potential advantages of ABS composite pipe include:

(a) Light weight.
(b) Long laying lengths (in some situations).
(c) Ease in field cutting.

Potential disadvantages of ABS composite pipe include:

(a) Limited range of sizes available.
(b) Subject to environmental stress cracking.
(c) Subject to rupture when improperly bedded.
(d) Subject to attack by certain organic chemicals.
(e) Subject to surface change affected by long-term ultraviolet exposure.

ABS composite pipe should be manufactured in accordance with one or more of the following standard specifications:

(a) "Acrylonitrile-Butadiene-Styrene (ABS) Composite Sewer Piping," ANSI/ASTM D 2680.
(b) "Solvent Cement for Acrylonitrile-Butadiene-Styrene (ABS) Plastic Pipe and Fittings," ANSI/ASTM D 2235.
(c) "Joints for Drain and Sewer Plastic Pipes Using Flexible Elastomeric Seals," ANSI/ASTM D 3212.
(d) "Elastomeric Seals (Gaskets) for Joining Plastic Pipe," ANSI/ASTM F477.

Polyethylene (PE) Pipe PE pipe is used for both gravity and pressure drains. Non-pressure PE pipe, primarily used for sewer relining, is available in nominal diameters from 4–48". PE fittings are available. Jointing is primarily accomplished by butt fusion or flanged adapters.

PE pipe is manufactured by extrusion of PE plastic material. Non-pressure PE is produced at this time in accordance with individual manufacturer's product standards. Potential advantages of PE pipe include:

(a) Long laying lengths (in some situations).
(b) Light weight.
(c) High impact strength.
(d) Ease in field cutting.

Potential disadvantages of PE pipe include:

(a) Relatively low tensile strength and pipe stiffness.
(b) Limited range of sizes available.
(c) Subject to excessive deflection when improperly bedded and haunched.
(d) Subject to attack by certain organic chemicals.
(e) Subject to surface change affected by long-term ultraviolet exposure.
(f) Special tooling required for fusing joints.

PE pipe is specified by material designation, nominal diameter (inside or outside), standard dimension ratios and type of joint. PE pipe should be manufactured in accordance with one or more of the following specifications:

(a) "Butt Heat Fusion Polyethylene (PE) Plastic Fittings for Polyethylene (PE) Plastic Fittings for Polyethylene (PE) Pipe and Tubing," ANSI/ASTM D 3261.
(b) "Polyethylene (PE) Pipe and Tubing," ANSI/ASTM D 2239.
(c) "Polyethylene (PE) Plastic Pipe (SDR-PR)," ANSI/ASTM D 3261.
(d) "Polyethylene (PE) Plastic Pipe (SDR-PR)," ANSI/ASTM D 2239.
(e) "Polyethylene (PE) Plastic Pipe (SDR-PR), Based on Controlled Outside Diameter," ANSI/ASTM D 3035.

Polyvinyl Chloride (PVC) Pipe PVC pipe is used for both gravity drains and storm sanitary sewers. Non-pressure PVC sewer pipe is available in nominal diameters from 4–27". PVC pressure and non-pressure fittings are available. PVC pipe is generally available in lengths up to 20'. Jointing is primarily accomplished with elastomeric seal gasket joints, although solvent cement joints for special applications are available.

PVC pipe is manufactured by extrusion of the plastic material. A wide variety of appurtenances are available including wyes, tees, saddles, elbows, increasers, plugs, couplings, manhole adapters, and adapters to other pipe material.

Non-pressure PVC sanitary sewer pipe is provided in three dimension ratios, DR 35, 41, and 51, which relate to pipe stiffness values, PS 46, 28, and 80 psi, respectively. Potential advantages of PVC pipe include:

(a) Light weight.
(b) Long laying length (in some situations).
(c) High impact strength.
(d) Ease in field cutting and tapping.

Potential disadvantages of PVC pipe include:

(a) Subject to attack by certain organic chemicals.
(b) Subject to excessive deflection when improperly bedded and haunched.
(c) Limited range of sizes available.
(d) Subject to surface changes affected by long-term ultraviolet exposure.

PVC pipe is specified by nominal diameter, dimension ratio, pipe stiffness and type of joint. PVC pipe should be manufactured in accordance with one or more of the following standard specifications:

(a) "Type PSM Polyvinyl Chloride (PVC) Sewer Pipe and Fittings," ANSI/ASTM D 3034.
(b) "Type PSP Polyvinyl Chloride (PVC) Sewer Pipe and Fittings," ANSI/ASTM D 3033.
(c) "Elastomeric Seals (Gaskets) for Joining Plastic Pipe," ANSI/ASTM F 477.

(d) "Joints for Drain and Sewer Plastic Pipes Using Flexible Elastomeric Seals," ANSI/ASTM D 3212.
(e) "PVC and ABS Injected Solvent Cemented Plastic Pipe Joints," ANSI/ASTM F 545.
(f) "Solvent Cements for Polyvinyl Chloride (PVC) Plastic Pipe and Fittings," ANSI/ASTM D 2564.
(g) "Standard Specification for Polyvinyl Chloride (PVC) Large Diameter Plastic Gravity Sewer Pipe and Fittings," ASTM F-679.

5. *Thermoset Plastic Pipe*

Thermoset plastic materials include a broad variety of plastics. These plastics, after having been cured by heat or other means, are substantially infusible and insoluble. Generally, thermoset plastic materials used in sewers are provided in two categories—reinforced thermosetting resin (RTR) and reinforced plastic mortar (RPM).

Reinforced Thermosetting Resin (RTR) Pipe RTR pipe is used for both gravity and pressure drains. RTR pipe is generally available in nominal diameters from 1—12", manufactured in accordance with ASTM standard specifications. The product is available in nominal diameters from 12–144" manufactured in accordance with individual manufacturer's specifications. In small diameters, RTR fittings are available. In large diameters, RTR fittings are manufactured as required. A number of jointing methods are available. Various methods of interior protection (e.g., thermoplastic or thermosetting liners or coatings) are available.

RTR pipe is manufactured using a number of methods including centrifugal casting, pressure laminating and filament winding. In general, the product contains fibrous reinforcement materials, such as fiberglass, embedded in or surrounded by cured thermosetting resin.

An example of an RTR pipe material is the "Insituform" process which is most commonly used for the rehabilitation of existing pipe materials. A polyester fiber felt tube with an impermeable layer on one side is impregnated with a liquid thermosetting resin. The tube is inserted through the existing pipe through an access point such as a manhole. Cold water is forced through the pipe, pressing the tube against the existing pipe. The water is then heated, curing the inserted tube into a pipe within a pipe. This "Insituform" process is available for sizes from 4" to several feet and can be used for various shaped conduits. The thermoset resins can be selected to meet the requirements for resistance to corrosion. There are no joints for infiltration to enter. The wall thickness is selected to meet structural requirements and is expressed as a standard dimension ratio, SDR. The existing waterway area is reduced by the tube wall thickness. The reduced area may be compensated in part by improved smoothness of the pipe. Potential advantages of RTR pipe include:

(a) Light weight.
(b) Long laying lengths (in some situations).

Potential disadvantages of RTR pipe include:

(a) Subject to strain corrosion in some environments.
(b) Subject to excessive deflection when improperly bedded and haunched.
(c) Subject to attack by certain organic chemicals.
(d) Subject to surface change affected by long-term ultraviolet exposure.

RTR pipe is specified by nominal diameter, pipe stiffness, lining and coating, method of manufacture, thermoset plastic material, and type of joints. RTR pipe should be manufactured in accordance with one or more of the following standard specifications:

(a) "Filament-Wound Reinforced Thermosetting Resin Pipe," ASTM D 2996.
(b) "Centrifugally-Cast Reinforced Thermosetting Resin Pipe," ANSI/ASTM D 2997.
(c) "Machine-Made Reinforced Thermosetting Resin Pipe," ASTM D 2310.

Reinforced Plastic Mortar (RPM) Pipe RPM pipe is used for both gravity and pressure sewers. RPM pipe is available in nominal diameters from 8–144". In smaller diameters, RPM fittings are manufactured as required. A number of jointing methods are available. Various methods of interior protection (e.g., thermoplastic or thermosetting liners or coatings) are available.

RPM pipe is manufactured containing fibrous reinforcements such as fiberglass and aggregates such as sand embedded in or surrounded by cured thermosetting resin. Potential advantages of RPM pipe include:

(a) Light weight.
(b) Long laying lengths (in some situations).

Potential disadvantages of RPM pipe include:

(a) Subject to strain corrosion in some environments.
(b) Subject to excessive deflection when improperly bedded and haunched.
(c) Subject to attack by certain organic chemicals.
(d) Subject to surface change affected by long-term ultraviolet exposure.

RPM pipe is specified by nominal diameter, pipe stiffness, stiffness factor, beam strength, hoop tensile strength, lining or coating, thermoset plastic material, and type of joint. RPM pipe should be manufactured in accordance with one or more of the following standard specifications:

(a) "Reinforced Plastic Mortar Sewer Pipe," ANSI/ASTM D 3252.
(b) "Reinforced Plastic Mortar Sewer and Industrial Pressure Pipe," ASTM D 3754.

C. Pipe Joints

A substantial variety of pipe joints are available for the different pipe materials used in drainage construction. A common requirement for

the design of all drainage systems, regardless of the type of pipe specified, is the use of reliable, tight pipe joints. A good pipe joint must be watertight, flexible and durable. Currently, various forms of gasket (elastomeric seal) pipe joints are used with many pipe materials. They generally can be assembled by unskilled labor in a broad range of weather conditions and environments with good assurance of a reliable, tight seal.

1. *Gasket Pipe Joints*

Gasket joints seal against leakage through compression of an elastomeric seal or ring. Gasketed pipe joint design is generally divided into two types; push-on joint and mechanical compression pipe joint.

2. *Push-on Pipe Joint*

This type of pipe joint uses a continuous elastomeric ring gasket compressed into an annular space formed by the pipe, fitting, or coupler socket, producing a positive seal when the pipe spigot is pushed into the socket. When using this type of pipe joint in pressure sewers, thrust restraint may be required to prevent joint separation under pressure. Push-on pipe joints (fittings, couplers, or integral bells) are available on nearly all pipe products mentioned.

3. *Mechanical Compression Pipe Joint*

This type of pipe joint uses a continuous elastomeric ring gasket which provides a positive seal when the gasket is compressed by means of a mechanical device. When using this type of pipe joint in pressure sewers, thrust restraint may be required to prevent joint separation under pressure. This type of pipe joint may be provided as an integral part of cast iron or ductile iron pipe. When incorporated into a coupler, this type of pipe joint may be used to join two similarly sized plain spigot ends of commonly used sewer pipe materials.

4. *Bituminous Pipe Joint*

This type of pipe joint involves use of hot-poured or cold-packed bituminous material forced into a bell-and-spigot pipe joint to provide a seal. The use of this joint is discouraged, since reliable, watertight joints are not assured.

5. *Cement Mortar Pipe Joint*

This type of pipe joint involves use of shrink-compensating cement mortar placed into a bell-and-spigot pipe joint to provide a seal. The use of this joint is discouraged for pressure drains because reliable, watertight joints are not assured. Cement mortar joints are not flexible and may crack if there is any pipe movement.

6. *Elastomeric Sealing Compound Pipe Joints*

Elastomeric sealing compound may be used in jointing properly prepared concrete gravity pipe. Pipe ends must be sandblasted and primed for elastomeric sealant application. The sealant, a thixotropic, two-part elastomer is mixed on the job site and applied with a caulking gun and spatula. The pipe joint, when assembled with proper materials and procedures, provides a positive seal against leakage.

7. *Solvent Cement Pipe Joints*

Solvent cement pipe joints may be used in jointing thermoplastic pipe materials such as ABS, ABS composite, and PVC pipe. This type of pipe joint involves bonding a sewer pipe spigot into a sewer pipe bell or coupler using a solvent cement. Solvent cement joints can provide a positive seal provided the proper cement is applied under proper ambient conditions with proper techniques. Refer to ASTM D 402 for safe handling procedures. Precautions must be taken to ensure adequate trench ventilation and protection for workers installing the pipe. Solvent cement pipe joints may be desired in special situations and with some plastic fittings.

8. *Heat Fusion Pipe Joints*

Heat fusion pipe joints are commonly specified for PE pipe. The general method of jointing PE pipe involves butt fusion of the pipe lengths. After the ends of two lengths of PE pipe are trimmed and softened to a melted state with heated metal plates, the pipe ends are forced together until they fuse, providing a positive seal. The pipe joint does not require thrust restraint in pressure applications. Trained technicians with special apparatus are required to achieve reliable watertight pipe joints.

9. *Mastic Pipe Joints*

Mastic pipe joints are frequently used for special non-round shapes of concrete pipe that are not adaptable for gasketed pipe joints. The mastic material is placed into the annular space to provide a positive seal. Application may be by trowelling, caulking, or by the use of preformed segments of mastic material in a manner similar to gaskets. Satisfactory performance of the pipe joints depends upon the proper selection of primer and mastic material, and on good workmanship.

10. *Sealing Band Joints*

External sealing bands of rubber made in conformance with ASTM C 877 are also used on non-circular concrete pipe. These elastomeric bands are wrapped tightly around the exterior of the pipe at the joint and extend several centimeters (inches) on each side of the joint. Sealing

against the concrete is achieved by a mastic applied to one side of the band.

IV. MAINTENANCE

A. Introduction

All storm drainage systems must be maintained. A well-maintained storm drainage system will be ready to convey the runoff from the next storm with minimal damage to the storm drainage facilities. A poorly maintained drainage system may not be able to function at its design conveyance and could be damaged by the runoff. The increases in potential repair costs and liability exposure are less obvious, but no less serious. Minor repairs can often prolong the service life of the facility, and can reduce the costs of future major repairs and/or replacement.

Because storm drainage systems function intermittently and seldom at full capacity, it is all too easy to defer maintenance activities. The storm drainage system must be actively maintained—it is too late to repair the damage from the last storm or to do preventive maintenance if storm clouds are again gathering over the basin. It is especially important to recognize that maintenance includes both scheduled (mowing, trash pickup, etc.) and unscheduled (erosion damage repair, etc.), and funds must be provided for both. Excellent discussions of the subject can be found in ASCE (1983) and WPCF (1989).

The owner of storm drainage facilities should establish a routine maintenance inspection program once the facility has been completed and placed in service. The inspections should be conducted on an annual or semi-annual basis, as well as following major storms. The inspections may be accomplished by visual means or by using a television camera, where applicable.

The inspection should be documented. Items to be recorded should include size and type of facility, date of inspection, location of facility, minor deficiencies, major deficiencies, and areas of possible future problems. The documentation should be kept current, and when any repair work has been accomplished, it should be recorded.

B. MAINTENANCE OBJECTIVES

A thorough drainage system maintenance program will provide for scheduled maintenance activities and will also accommodate necessary unscheduled work. Staff from the maintenance department should be involved in all aspects of a drainage system, from planning and design through to construction, if maintenance considerations are to receive adequate attention. The general goals of a maintenance program should include the following:

(a) Participate in drainage project planning and in design review to facilitate maintenance activities.

(b) Participate in construction progress meetings to determine if maintenance-oriented facilities are being built as called for in the design.
(c) Inspect facilities regularly to monitor their effectiveness and need for repairs.
(d) Prevent drainage systems from falling into visual disrepair. Aesthetic considerations are important within the community. A respectable-looking drainage facility is less likely to attract vandalism and garbage dumping, and will help maintain adjacent property values.
(e) Reduce life-cycle costs through effective design review, timely maintenance activities, documentation of crew and equipment productivity, and analysis of repair costs and longevity.
(f) Repair deteriorated facilities before major damage or failure occurs.
(g) Have drainage systems repaired, cleaned, and ready to function before the next rainy season arrives.
(h) Repair and maintain facilities as necessary to insure they are capable of operating at full design conveyance.

C. Life-cycle Stages of a Storm Drainage System

All improved storm drainage systems pass through three stages in their life-cycle. Those stages are:

(a) Design and Construction.
(b) Drainage Service.
(c) Rehabilitation/Replacement.

If a drainage system includes many structures and man-made features, the stages are quite separate and distinct. The stages are less obvious and less important for streams that have been only slightly modified by man. The higher the degree of improvement to be done to a drainage channel, the more imperative it is that maintenance personnel are involved in every stage.

The design and construction stage is short but it is a period of much activity with far-reaching affects. It is the beginning of the life-cycle for the drainage system. The maintenance-oriented decisions made during this stage will dictate much of what happens during the other two stages.

The drainage service stage will be long and uneventful if maintenance concerns are given full consideration during design. During the drainage service stage the drainage system should function as designed. Maintenance activities are directed at extending the service life as much as possible.

All drainage systems will eventually pass through a rehabilitation stage, or will have features that are better replaced than rehabilitated. Facilities that are storm-damaged or simply worn out will need repair. As long as the hydrology and hydraulics are still valid the system can be rehabilitated and returned to the drainage service stage. The more comprehensive and maintenance-oriented the original design the more likely it is that the system will need rehabilitation rather than complete re-design and reconstruction. The life-cycle stages will be discussed in

each of the following sections regarding maintenance of different drainage systems.

D. Maintenance of Open Channel Drainage Systems

Many factors make open channel drainage systems more desirable than other systems. Those same factors need careful attention from maintenance personnel during the three life-cycle stages.

1. Design and Construction Stage

(a) Access—Vehicle access is vital to the maintainability of a drainage system. Ramps leading into channels and/or all-weather trails paralleling the system are frequently multi-purpose designs. Be sure access ramps and trails have traffic control barriers to keep unwanted traffic from using the trails while still allowing pedestrian movement.

(b) Side slopes—Grass-lined channels must have slopes that are steep enough to drain toward the channel and yet are gentle enough to allow vegetation to establish and to permit mowing and clean-up activities.

(c) Vandalism—Drainage facilities can be attractive nuisances and can be damaged by those who use the area. Preventive measures may be necessary to keep graffiti off walls, to keep rock riprap from being relocated, or to keep gabion baskets from being cut open.

(d) Trickle channels—Base flow erosion damage continues day after day. The cumulative effect can be dramatic. If the soils are erodible it may be necessary to install a trickle channel to halt the erosion. Pay attention to the potential for erosion immediately outside the trickle channel during intermediate runoff events. Reach agreement with pertinent regulators, in advance of construction, that channel maintenance will not require a special permit, such as a "404" permit under the Clean Water Act.

(e) Localized erosion—There are several locations that can suffer erosion and subsequently need increased maintenance work. Proper design will reduce the likelihood of erosion problems. A practical review of the project plans may reveal the need for additional erosion protection in the following places:
   (1) All transitions, such as changes in cross-section or changes in channel lining material.
   (2) At the outside of curves where flow velocities are higher.
   (3) At the outlet of all tributary storm sewer pipes.
   (4) On the bank opposite all tributary pipes and channels.
   (5) Downstream from drop structure energy dissipation basins.
   (6) Downstream of bridges and box culverts.

(f) Toe protection—Localized scour or general degradation can quickly lower the bottom of a channel. Erosion protection facilities must have deep toe protection or they can fail by being undermined.

(g) Rundowns—All drainage systems have many small capacity tributaries. Runoff events can damage these tributary connections as well as the main channel if the connections are not built to withstand the erosion impact.

(h) Trash racks—Normally, these structures do exactly what they are designed to do—catch debris. The bars should be spaced to allow small

debris to pass through yet catch large material. Arrange the bars to facilitate cleaning and to allow the debris to float out of the way as the water level rises.

(i) Sediment traps—If called for in the design, they will certainly need regular silt removal to protect the downstream facilities. Sediment traps effectively reduce downstream maintenance needs.

2. *Drainage Service Stage*

(a) Mowing—In urbanized areas the drainage channel should be mowed often enough to control weeds and to show community responsibility. For native grass vegetation in a semi-arid climate three to six cuttings per year is satisfactory.

(b) Debris control—Debris blockage at drainage structures often contributes to flooding problems. Trash racks and debris traps help reduce the problem if they function properly and are regularly cleaned. Regular debris removal along the length of the drainage system also helps. This should include trimming and thinning of trees if they encroach on the drainage channel or if they have become overgrown.

(c) Inspection—An inspection at least annually of drainage facilities will detail long-term changes in the system and will highlight needed maintenance work. Inspections should also be done following major storm events.

(d) Silt removal—Some silt accumulation in stilling basins and around channel obstructions is inevitable, and is harmless in limited amounts. Silt should be removed if it is severe enough to alter the water surface or affect the function of drainage facilities such as drop structures. Silt accumulations can also cause trouble by supporting undesirable or obstructive vegetation. Remember that suitable silt disposal sites can be a problem (due to contaminants) and should be evaluated during design.

(e) Trail repair—An annual effort to repair damaged trail sections will result in guaranteed maintenance access and better pedestrian use. The best time for repairs is right after the cold/rainy season.

3. *Rehabilitation Stage*

With regular inspection reports a drainage maintenance department will know when a drainage system is in need of repairs. If the problems are repaired promptly the facility can be returned to service with little threat of further damage or failure. Listed below are many of the typical problem areas that signal the need for rehabilitation.

(a) Hard-lined trickle/low flow channels—Local undermining of the structure or secondary channel erosion parallel to the main channel.

(b) Soft-lined trickle/low flow channels—Random bank failure and bottom degradation that is unsafe or threatens other improvements.

(c) Tributary channels and pipe outlets—Erosion from the receiving channel leading back to the tributary outlet and/or erosion under the outlet structure.

(d) Drop structures and grade control structures—Erosion damage in and around the energy dissipation basin and around the outside edges of

the structure. Physical damage can occur to the structure in the form of uplifted or depressed concrete, broken gabion baskets, and displaced riprap.

(e) Channel banks—Channel bottom and sides should be maintained to their original slopes. Bank protection such as riprap, slope paving, or retaining walls can be undermined by local scour or general degradation if not "toed-in" deep enough. Grass-lined banks can lose vegetative cover and can suffer spot erosion that may quickly worsen.

E. Maintenance of Piped Drainage Systems

Right-of-way constraints frequently dictate use of a piped drainage system, which in turn create particular maintenance constraints.

1. *Design and Construction Stage*

(a) Access—Manpower and equipment access for the length of the piped system must be available. Publicly owned right-of-way or easements are normally sufficient (be sure the easement connects to a public right-of-way and is wide enough for maintenance activities). The easement language must be restrictive enough to prohibit undesirable activities on the land surface.

(b) Erosion protection—The inlets and outlets to piped systems are subject to high velocities. Adequate protection will usually take the form of riprap aprons, energy dissipation structures, or concrete headwalls, wingwalls and aprons. Steep earth slopes at inlet and outlet transitions frequently need short walls to hold the soil in place.

(c) Trash Racks—This is one of the most frequent problem areas. The design should consider the potential debris sources upstream. Designers should assume at least 50% blockage of a trash rack when designing for the maximum storm runoff. Another rule of thumb requires a trash rack to have four times the clear opening area of the pipe being protected.

(d) Manholes—Most local governments have their own requirements for spacing. Manholes need to be accessible in all weather conditions. Be sure access is available to all pipes of a multi-barrel system. Drop manholes can be especially difficult when designing for adequate access and safety.

(e) As-built drawings—It is an absolute necessity to obtain as-built drawings of the completed project.

2. *Drainage Service Stage*

(a) Curb inlet cleaning—Because of their location and shape inlets often trap sediment and debris. They should be cleaned twice a year to insure their proper function. If only one cleaning is possible it should occur prior to the rainy season.

(b) Debris control—Trash racks should be cleaned regularly to keep accumulations from forming. In-pipe debris should be removed if it is large enough to create a flow obstruction.

(c) Overflow channel maintenance—If the pipe system was designed with a surcharge or overflow channel it deserves occasional attention. It

must be kept clear of debris and excessive vegetation. In general, it should be maintained as an open channel to be ready to function when called upon.

(d) Inspection—A regular in-pipe inspection of piped drainage systems will detail long term changes and will point out needed maintenance work such as debris removal or joint patching. Special attention is necessary to insure the safety of the inspection team if the pipe is long. Small pipes and pipes that carry continuous flow can be viewed with automated equipment. Inspections should be done following major runoff events. Inlet grates should be checked for clogging, and catch basins and pipes for sediment/waste blockage.

3. *Rehabilitation Stage*

Typical problem areas that can signal the need for rehabilitation of a piped system include:

(a) Inlet and outlet structures—Local erosion due to high velocities, lack of protection, or transition turbulence.
(b) Trench backfill—Subsidence of the trench, which can result from poor initial compaction or from pipe or joint failure. Earth settlement around manholes is a frequent indicator of compaction problems.
(c) Pipe joints—The first sign of problems in the system shows at the pipe joints. Spalled concrete, cracks, distorted pipe geometry, backfill movement, and water inflow occur at the joints and are precursors of greater problems to come.

V. SUMMARY

The choice of a particular material depends upon a number of variables. The best choice is the one that yields the best performance over the project life cycle. Some products have a much longer useful life in a particular environment than others. The engineer must evaluate material longevity based on a realistic appraisal of how long the stormwater conveyance or system will be necessary with respect to projected development growth rates and master planning priorities established by the community. The life-cycle cost analysis will insure the proper material specification after the factors of capital cost, annual maintenance costs, and life expectancy are applied. Regionally specific concerns, such as material availability, will influence the selection. Finally, the forces that degrade storm drainage systems vary from one location to another, and prudent evaluation of these factors is advised.

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CHAPTER 14

STRUCTURAL REQUIREMENTS

I. INTRODUCTION

Storm drainage structures vary from relatively conventional storm sewers and culverts to unique facilities designed for specific purposes. Structural details of all of the facilities that might be included in drainage and flood control structures are beyond the scope of this Manual, though the design engineer must, nevertheless, be cognizant of them.

It is assumed that the design of the more conventional structures will be completed by the hydraulic engineer, after appropriate consultation with a geotechnical engineer. Specific guidelines or references are provided to facilitate this effort. It is also assumed that unique, large, or complex structures will probably be designed by structural and/or geotechnical engineers. In such cases, stormwater facility designers must take an active role, providing guidance or input in the structural design of all hydraulic facilities. The hydraulic engineer should be responsible for identifying the critical design conditions and computing the loads on the various members of the structure.

II. STRUCTURAL DESIGN PROCESS

The structural design process consists of the following steps:

(a) Identify critical design condition(s).
(b) Determine loadings in the critical design condition(s).
(c) Perform a stability analysis on the overall structure.
(d) Select and design structural members.
(e) Prepare a structural plan and details.

A structural engineer unfamiliar with drainage facilities can be of significant assistance only in the final two steps. The first four steps of the structural design process are addressed in detail in the remainder

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of this chapter. The fifth step, the preparation of plans and details, is covered in numerous texts and manuals and is not addressed here.

III. PROJECT LIFETIME FOR STRUCTURAL DESIGN

It should not be assumed that the structural elements of a facility need to be designed for a service life equal to the design return period for the facility itself, and it is important that a designer distinguish between several different "time intervals" frequently associated with a project.

"Project life" or "design life," also discussed in Chapter 13, are terms that refer to the length of time a facility will last, after which it may require replacement. This lifetime is generally based upon factors that occur gradually, such as wearing of equipment or linings, corrosion of metals, and deterioration of concrete.

Since the lengths of time within which a facility may need repair due to these processes are all different, they are frequently aggregated into the terms "life cycle" or "economic life." These refer to a time period after which it would be more economical to replace than repair the facility. Commonly selected economic lives for drainage and flood control projects are 35, 50, or 75 years, despite the fact that many projects provide much longer service. It is important to note that the project life is essentially independent of a project's design return period. Projects designed to convey a five-year flood or a 100-year flood may each have a project life of 35 years.

The project design return period (for hydraulic performance) will establish the flow conditions—and therefore the loading conditions—for which the structures will be designed. When greater loads occur, it must be assumed that the structure could fail, unless specifically designed for overload conditions. Note, however, that structural failure is generally unacceptable in any but the most rare events. This is particularly true if a failure would result in damage, injury, or death, as in the case of a failure of a dam or levee.

The project design life (for structural design) must be selected carefully for each project, and an analysis must be made of the critical design conditions and the failure mechanisms for specific structures. The following guidelines should be considered:

(a) The project design life should, in many cases, be at least as great as the project's economic life. A facility that an owner would plan to replace at fifty-year intervals should not experience structural failure more frequently (Owners should, however, be reminded that failure due to hydraulic events is based upon probability and not upon elapsed time).

(b) If structural failure could be catastrophic, such as the failure of a large dam, then the structures should be designed to survive very rare events, such as the Probable Maximum Flood. It should be noted that com-

pletely unsatisfactory hydraulic performance can occasionally be tolerated, if the structural performance remains satisfactory.

IV. ESTABLISHMENT OF DESIGN CONDITIONS

A. General

The hydraulic selection and sizing of a drainage facility requires the identification of at least one "critical design condition." For purposes of this discussion, a "design condition" is a combination of loads that could be exerted simultaneously on a structure or any of its members. A "critical design condition" is one in which the loads create a maximum net force in a single direction on a structure or any of its members. Any structure will have a large number of design conditions, from which must be selected the few critical design conditions to be addressed in the design process. The engineer must consider all conditions that could occur during the life of the structure. He must identify conditions that could or could not occur simultaneously, and he must consider which simultaneous conditions might cause increased loads and which loads would tend to reduce or cancel other loads.

As part of the process of identifying critical design conditions, the engineer should consider mechanisms through which failure might occur, such as rare flow events or excessive scour.

The following sections describe some conditions that could constitute critical design conditions (it should not be assumed that all possible critical design conditions are included). While each possible condition should be considered for each structure, the designer's experience should allow him to quickly discard certain possibilities as inappropriate for specific situations.

3. Flow Conditions

Since most drainage and flood control structures are designed to convey or control water, flow conditions are frequently the most readily predictable design conditions. Each of the following should be considered:

(a) No Flow—The absence of flow is a relatively common condition that may not, in itself, create excessive forces, but that may be the condition under which other loads reach their maximum values.
(b) Low Flow—Such a condition would rarely create a critical design condition, but it should be considered.
(c) Design Flow—Design flow generally is a large flow with high velocities and, in most cases, will represent a portion of one or more critical design conditions.
(d) Rare Flow—Most drainage and flood control structures have design flows with return periods that may range up to 100 years. However, many could also be subject to much larger flood events. The engineer

should consider the potential for structural failure as a result of such events. If they could cause significant additional damage or loss of life, the facility should be designed accordingly.

C. Groundwater Conditions

If all or part of any structure or member will be below groundwater level, hydrostatic pressures will be exerted, as if the structure were partially or completely submerged in a body of water. This can result in uplift forces that tend to lift the structure out of the ground and reduce the sliding friction resistance at boundaries between the structure and the soil or rock. The groundwater pressures also produce loads within the structural elements, such as bending moments and shear stresses within a retaining wall that has groundwater behind it.

When a designer selects the groundwater level for the design of a particular structure, he must consider both the typical groundwater conditions and the fluctuations in level that might be anticipated due to seasonal variations, flood events, and other applicable factors. The permeability of the soil and rock strata will affect the magnitude and timing of the expected fluctuations.

Careful consideration should be given to the adequacy and effectiveness of design details intended to either encourage groundwater flow through a structure or to prohibit groundwater flow into a structure. For example, weep holes and other drainage behind retaining walls should have adequate capacity to maintain the groundwater levels assumed in the design. Water stops and sealed joints used to prevent seepage through structural elements should be strong enough to withstand the expected hydrostatic pressures, and should be able to accommodate any expected differential movements caused by settlement or loads.

D. Adjacent Earth Conditions

All structures are affected to varying degrees by earth loadings. The loadings to impose on a structure after backfilling are generally determined by a geotechnical engineer. His recommendations must be based upon analyses of the soils to be used for backfilling, which may be either earth existing at the site or imported material.

When a hydraulic structure reaches various stages of partial completion, it could have loading conditions imposed upon it that stress its members either to a greater degree or in a different direction than would occur after backfilling. In particular, flexible conduits and sloping (or warped) wing walls may require special supports until backfilling is completed.

Many hydraulic structures can be subjected to critical design conditions resulting from either excessive erosion or scour, or from the deposition of sediment. While such conditions generally last for only a short time, they are usually associated with large flows or rare flood

events, with the critical hydraulic loads occurring simultaneously with a removal of resisting or supporting soil.

It is important that the hydraulic engineer make a careful evaluation of two scour conditions. The first is scour that could reasonably be anticipated during high flow conditions. It is generally prudent to assume some level of erosion in scour-prone areas adjacent to a structure, even when great care is given to erosion prevention.

The second condition to evaluate is one that might occur when flows exceed the design flow. The engineer should consider whether extreme scour could lead to a complete structural failure, and whether it would cause damage or loss of life. In such cases, it may be appropriate to insure that a structural failure would not occur.

E. Superimposed Loadings

Superimposed loadings result from movable objects, including pedestrians, automobiles, trucks, railroad locomotives and aircraft. Maintenance equipment, such as mowers or dump trucks, also create superimposed loads. In unusual conditions, unexpected construction of future roadways, buildings or other facilities adjacent to a structure could create superimposed loads.

F. Construction Conditions

Hydraulic structures are frequently exposed to critical design conditions during construction. The hydraulic and geotechnical engineers must advise the contractor when this is likely to occur, so that damage to a partially completed structure can be minimized. The standard contract language that makes a contractor responsible for protection of the site during construction does not eliminate the engineer's responsibility to insure that a structure's integrity can be maintained during construction. If a specific structure is uniquely vulnerable to damage or failure in specific stages of construction, the designer should consider ways to reduce the vulnerability and should advise the contractor accordingly. Loadings of particular concern include hydrostatic loads behind walls (where soil backfill would eventually be placed), earth loadings due to excessive sediment deposition or sliding banks, and uplift created before backfill is placed.

Finally, any member that is to be moved into place must be designed and constructed so that it can be lifted and transported without damage. Supporting requirements should be identified for items such as pipes, which might be lifted by cables or slings.

G. Documentation of Critical Design Conditions

Once the critical design conditions are established, the hydraulic engineer should identify and record the following information for each condition:

(a) Finished earth grades adjacent to the structure (including scour predictions).
(b) Design water surfaces.
(c) Design flow velocities.
(d) Ground water levels.
(e) Seepage conditions.
(f) Superimposed loads.
(g) Construction stages and loadings.

V. DETERMINATION OF LOADS

A. General

The design of a drainage or flood control structure requires the determination of the composite forces on the overall structure and on each individual member of the structure. These composite forces result primarily from fluid pressures, earth pressures, and superimposed loads. Miscellaneous loadings such as wind or earthquakes can have effects on certain types of structures. Such loads are beyond the scope of this Manual, and may be determined with the aid of manuals or textbooks on fluid and soil mechanics or structural design.

B. Hydraulic Loads

Hydraulic loads on structures are characterized as either hydrostatic, resulting from fluid pressures, or hydrodynamic, resulting from fluid movement. Most structures described in this Manual are affected by both hydrostatic and hydrodynamic forces. In many cases the hydrostatic forces will represent the dominant load on a structure.

Hydrodynamic forces may be significant if flow through or adjacent to the structure experiences an abrupt change in elevation, velocity, or direction. Hydrostatic pressures act equally in all directions in both static and moving water. Hydrostatic pressure may be determined by:

$$P = \gamma h \tag{14-1}$$

where:

P = pressure, in pounds per square foot
γ = unit weight of water, 62.4 pounds per cubic foot
h = vertical distance to the water surface, in feet

Hydrostatic pressures act perpendicular to the surface in contact with the water, and increase in proportion to the water depth. Structures designed to produce significant changes in water surface elevations or pressures may experience significant differential hydrostatic loads.

Hydrostatic loads that tend to reduce the composite loads on structures should be considered only if it is certain that those forces will

exist in the design condition. Proper structural design practice requires that intermittent forces, or forces that may cease to exist, must be neglected to the extent that they would tend to reduce or offset other loads.

The hydrodynamic forces most commonly of concern in structural design result from friction and momentum. Water flowing at a uniform depth (or pressure, in a closed conduit) and at a uniform velocity exerts a force on the walls of the conveying facility or on obstructions to the flow. The force acts in the direction of the flow and is frequently negligible in comparison to other forces on a structure. Conversely, water flow that changes in depth, velocity or direction can exert a relatively greater force, which is commonly referred to as an impact force or momentum force. The magnitude of a momentum force is determined by use of the momentum equation. Newton's second law of motion states that force equals the time rate of change of momentum. The resultant of forces on an element of water is determined by Equation 14-2:

$$F = \rho Q \Delta V \qquad 14\text{-}2$$

where

F = the net force on an element of water
ρ = the density of water, 1.94 slugs per cubic foot
Q = the flow rate, in cubic feet per second
ΔV = change in water velocity in the direction of the force, in feet per second

Since F is the force acting on the water element that causes the momentum change, its reaction is the force exerted on the structure by the moving water. Equation 14-2 is generally solved in vector form and can be simplified for one dimensional flow as follows:

$$F_x = \rho Q\,(V_{1x} - V_{2x}) \qquad (14\text{-}3)$$

where the subscript x implies a directional vector, and where V_{1x} and V_{2x} are the velocities on each side of the object subjected to the hydrodynamic force.

If the flow is two dimensional or three dimensional, Equation 14-3 can be written to determine forces parallel to the other axes. The determination of hydrodynamic forces begins with the computation of flow depths, velocities, and directions. Complex flow patterns in a hydraulic structure may create complex forces on the structural members.

C. Earth Loads

Hydraulic structures generally are affected by earth loads and, in turn, impose a load on the soil or rock adjacent to the structure. De-

terminations of earth loads should be based on a soils analysis made by a geotechnical engineer. The hydraulic engineer and the geotechnical engineer should together determine the soil parameters and loads required for a particular situation. These typically include the unit weight of the soil or soils involved; active, at-rest, and passive earth pressures; design groundwater levels; allowable bearing capacity; and coefficients of friction.

In general, earth loads on structures are the sum of the effective soil stress (the pressure exerted on the structure by the soil particles) and the groundwater pressures (the hydrostatic pressures exerted on the structure by the pore water in the soil).

The vertical effective soil stress (the soil stress acting downward on a horizontal plane) is the total downward stress minus the groundwater pressure acting upward at that point. For the simple case of a point at depth z_1 below a horizontal soil surface, and at a depth z_2 below the groundwater level, with z_2 less than z_1, the vertical effective stress is given by:

$$\bar{\sigma}_z = \gamma_t Z_1 - \gamma_w Z_2 \tag{14-4}$$

where

$\bar{\sigma}_z$ = the vertical effective stress, lbs/ft^2
γ_t = the total unit weight of the soil, lbs/ft^3
γ_w = the unit weight of water, lbs/ft^3

For cases where building or wall foundations or underground structures such as buried tanks are involved, the calculations are somewhat more complicated and distributions of stresses within the soil must be determined. However, the principle is still the same.

The horizontal effective soil stress (the soil stress acting laterally on a vertical plane, such as the back of a retaining wall or the vertical wall of an underground structure) is normally estimated by multiplying the vertical effective soil stress by an earth pressure coefficient, as in Equation 14-5.

$$\bar{\sigma}_H = K\sigma_v \tag{14-5}$$

where

$\bar{\sigma}_H$ = the horizontal effective soil stress, lb/ft^2
K = the earth pressure coefficient
$\bar{\sigma}_v$ = the vertical effective soil stress, lb/ft^2

The three commonly used earth pressure coefficients are the active, at-rest, and passive coefficients.

The active earth pressure is appropriate for estimating the earth pressure on a wall that moves away from the soil mass, and corresponds

to a state of failure in the soil. The active earth pressure is often used for the pressure behind a retaining wall. However, the factors of safety must be high enough to insure that a structure is safe. The design of a structure with active earth pressures and factors of safety near unity will result in a design very close to a state of failure.

The at-rest earth pressure would result if there were no horizontal movement of the surface. It is the appropriate pressure for rigid walls, such as the walls of underground concrete tanks. Some designers also use at-rest pressures to design retaining walls, since it is likely that, unless the wall moves repeatedly over its lifetime, the earth pressure behind the wall will build up to active earth pressure. If at-rest pressures are used in the design of a retaining wall, it is reasonable to use lower factors of safety, because the at-rest pressures are reasonable estimates of the maximum pressures the wall might experience.

The passive pressure is the maximum pressure on a surface that is pushed or moved into a soil mass in a horizontal direction. Passive pressure is often used as a component of resistance to sliding of retaining walls or other structures. Some cautions are appropriate relative to the use of passive pressure. First, it typically takes several inches of wall movement into the soil to fully develop the passive pressure, while it takes only a fraction of an inch of wall movement away from the soil to reduce pressure to active conditions. Unless several inches of movement can be tolerated, full passive pressure should not be included as a resisting force in design calculations, and full passive pressure values should be divided by factors of between two and three to arrive at design values. Second, the soil surface at the toe of the retaining wall must extend a significant distance outward from the wall for the passive pressure to be fully developed.

Some designers use "equivalent fluid pressures" to estimate horizontal earth pressures rather than using earth pressure coefficients applied to the vertical effective stress. In this method, the soil is assigned an "equivalent unit weight" and the horizontal pressure at any point is calculated as the hydrostatic pressure that would exist at a point below the surface of a fluid with that unit weight. Equivalent fluid pressures can be selected to represent either the effective soil stress or groundwater pressure. In the former case, any existing groundwater pressure would be added to the equivalent fluid pressure. When the equivalent fluid pressure method is applied, care must be taken so that the resulting pressures reasonably approximate the actual combination of effective soil stress and groundwater pressure.

Values of bearing capacity are provided by geotechnical engineers in two different formats. The designer must understand whether the value provided is an ultimate bearing capacity, which would not include a factor of safety, or an allowable or design bearing capacity, which has been reduced by an appropriate factor of safety or by consideration of an allowable settlement, if appropriate.

When coefficients of friction are used in an evaluation of sliding, it is important that consideration be given to both the potential for sliding within the soil itself and the potential for sliding along the boundary between the soil and the structure (e.g., concrete or steel). The coefficients of friction are different for the two cases, and the more critical should be determined. It should also be noted that the coefficients of friction apply only to forces resulting from effective soil pressures, and not to those resulting from groundwater pressures. The effects of groundwater pressures, however, must be considered when sliding friction forces are calculated, since they would have an uplift component that would reduce the structure weight. This would, in turn, reduce the friction force that would resist sliding.

D. Groundwater Loads

Groundwater pressures on drainage structures are frequently overlooked, but can produce significant and sometimes unexpected forces. For the purposes of this discussion, groundwater is defined as water able to move freely between soil particles, and the groundwater level is defined as the elevation to which water would rise in an open, uncased hole. These definitions differentiate between groundwater and the normal soil moisture present in almost all soils.

Since drainage structures are frequently built in low-lying or channel areas, they may extend underground into the natural groundwater. The natural soils in valley areas are frequently coarse and permeable, thus allowing free movement of groundwater. In addition, drainage facilities built in excavations are frequently backfilled with relatively coarse, freely draining material.

Pressures can not only be produced by the natural groundwater in the vicinity of a structure, but also from the sudden flooding or saturation of adjacent soil as a result of high flow through (or overtopping) a structure.

Since groundwater pressures are related to soil pressures, they are sometimes assumed to be part of the soil pressures in computations. This practice may be an oversimplification. When excessive soil pressures cause a deflection in structural members, the interlocking of particles within the soil matrix will frequently result in a "relaxation" of the load, limiting further deflections. Groundwater pressures will not relax, since the water can flow into the void created by a structural deflection and maintain the loading.

Groundwater pressures are identical to the hydrostatic pressures described in the previous subsection. When the groundwater conditions are relatively static and the level is approximately horizontal, groundwater pressures may be computed using Equation 14.1, which implies that equal pressures occur at equal depths. When a structure is immersed completely or partly in groundwater, it may act as a floating vessel, with the upward force equal to the weight of the displaced fluid.

If the upward buoyant force exceeds the weight of the structure and the weight of any soil, water, or other material, as may occur in the case of an empty tank or pipe, flotation can occur.

Groundwater moving around or beneath a structure, such as a dam or drop structure, moves from an area of relatively higher to lower ground water level. In such a case, the groundwater level cannot be assumed to be horizontal, and groundwater pressures along seepage paths are reduced in the direction of the seepage. "Lane's Weighted Creep" procedure, (U.S. Department of Interior 1960), provides a simplified method of computing groundwater pressures along a seepage path. The procedure also provides a mechanism for evaluating various methods of reducing groundwater pressures and seepage by adding cut-off walls or modifying the structure's dimensions. If significant groundwater pressures along a seepage path are anticipated, more detailed analyses are merited.

Accurate determination of groundwater pressures for structural design requires a careful evaluation of the extent to which the pressure can build up outside of a structure. In some structures, full groundwater pressures are allowed to develop and the structures are designed to resist them. In other structures, exterior drainage facilities are constructed to insure that the groundwater pressures cannot exceed the levels for which the structures are designed.

E. Superimposed Loads

Superimposed loads result from the weight and impact of vehicles over and adjacent to structures. The vehicles that could create the loads have legal limits that establish maximum wheel or axle loads. When the vehicle comes in contact with the structure, such as a bridge, inlet, or manhole vault, the load is equal to the maximum number of wheels, axles, or vehicles that could simultaneously be located over the structure. Loads are generally increased to represent impact if the vehicles are moving. The designer must determine the location of the load or combination of loads that would create the critical design condition for the structural member. For example, it is common to compare a single wheel in the center of a span to two adjacent wheels located as close to the center of the span as possible. The wheel load could be assumed to be a point load in a relatively large structure, or to be spread over the area of the wheel on smaller structures. For a further discussion, see Section VIII.

VI. STABILITY ANALYSIS

A. General

The initial step in the structural design of a drainage facility is the analysis of the overall structure's stability under the critical design

conditions. The structure itself, and the resisting forces acting on it, must be sufficient to insure that the structure can resist overturning, sliding and uplift. In addition, the structure's load on the underlying soil must not exceed the allowable bearing pressure.

B. Overturning

Many structures are exposed to loads that exert little or no overturning forces. However, in structures such as retaining walls, drop structures, or dams, overturning is frequently a principal concern.

Overturning is evaluated by computing the forces that would tend to overturn the structure, and the forces that would resist overturning. The point at which overturning would occur is determined, and moments are computed about that point. Moments that resist overturning should exceed those causing overturning by a factor of safety of 1.5 or more. Figures 14.1 and 14.2 illustrate the typical forces that would act on a small drop structure in an overturning evaluation, under two different conditions.

The designer who has difficulty providing an adequate factor of safety against overturning for a specific structure has two principal alternatives available. He may modify the structure to increase the magnitude of the resisting moments, or he may increase the distance, and thus the moment arm, to the point about which overturning would occur.

Resisting moments are increased through an addition to the structure weight or an extension of underground footings, which would mobilize a greater weight of resisting soil. Anchors into the soil or rock can also be constructed to add resisting moments. The resisting moment arm is generally increased by an extension of the structure length or foundation.

C. Sliding

Structures subjected to large horizontal forces, including dams, drop structures, and retaining walls, may slide along the earth foundation. The sliding could occur along a plane between adjacent layers of soil or rock, or along a plane between the structure and the underlying soil. The determination of a structure's stability in sliding requires an evaluation of the forces causing sliding and those resisting sliding in the critical design conditions.

Forces causing sliding are generally hydrostatic forces, hydrodynamic forces, and active earth pressures. Forces that resist sliding usually include the passive earth pressures of the downstream soil and the friction force developed between the structure and its soil or rock foundation. Figures 14.1 and 14.2 also illustrate the forces acting on a small drop structure subjected to sliding. The friction force is computed as follows:

$$F = \mu_F W \tag{14-6}$$

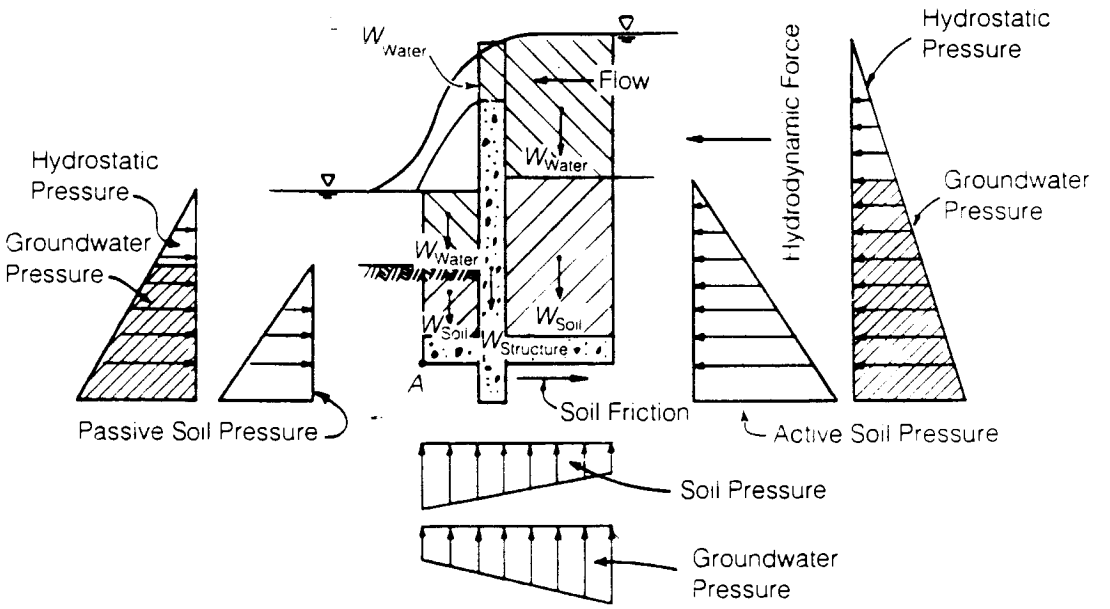


Figure 14.1—Evaluation of drop structure for overturning about toe (point A) and sliding along structure base during high flow.

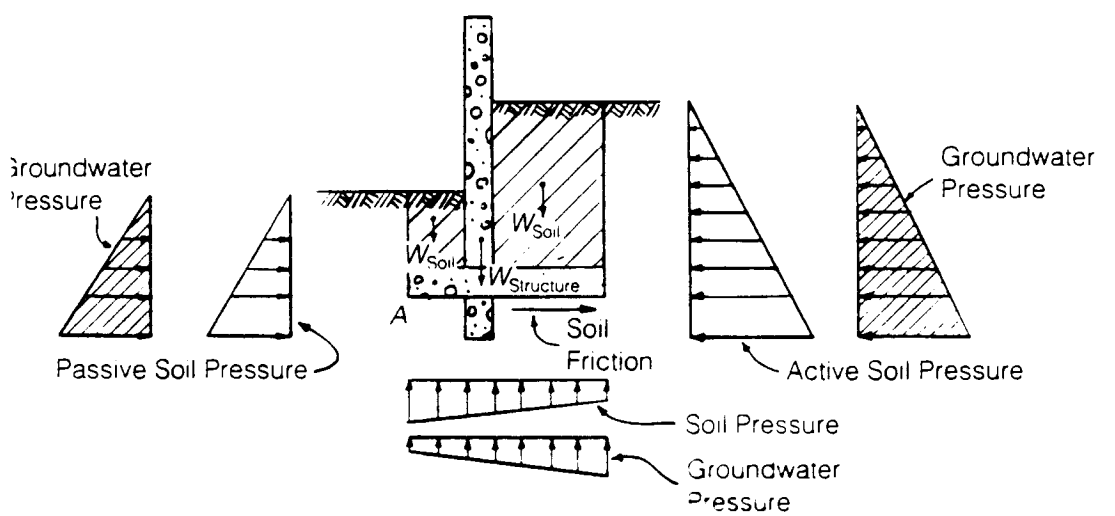


Figure 14.2—Evaluation of drop structure for overturning about toe (point A) and sliding along structure base after passage of high flow.

where

F = force, in pounds, required to initiate sliding
μ_F = coefficient of friction between soil layers or between the structure and the soil
W = the downward force, in pounds, the structure exerts on the soil

Values of μ_F must be determined based upon soils investigations. The downward force, W, includes the weight of the structure as well as any material, including earth, water, or vehicles in or above the structure that tend to increase the downward force on the soil foundation. The weight of water or vehicles should be included only when it is certain that they would be present during the design condition under consideration.

The critical design conditions under which sliding should be evaluated require careful identification. As described earlier, the critical design condition with respect to sliding may occur immediately after a high flow, as illustrated in Figure 14.2, since downstream erosion may tend to reduce resisting forces and high groundwater pressures may tend to reduce the value of W.

The earth pressures tending to create sliding are assumed to be active earth pressures, since the structure would tend to slide away from the earth. Conversely, earth pressures tending to resist sliding are assumed to be passive earth pressures, since the structure would be forced into the earth. Resisting passive pressures may be reduced by a factor of safety to allow for failure that might result from an inadequate extent of resisting soil. After the forces are determined, a structure's stability is computed by determining its safety factor against sliding using equation (14-7):

$$F.S._S = \frac{F_R}{F_S} \tag{14-7}$$

where

$F.S._S$ = factor of safety against sliding
F_R = summation of forces resisting sliding including the friction force of Equation (14-6)
F_S = summation of forces creating sliding

Generally, a safety factor against sliding of 1.5 or greater is considered adequate, although values as low as 1.25 may be satisfactory in some circumstances. A geotechnical engineer should be consulted before a safety factor of less than 1.5 is adopted.

When a designer has difficulty achieving satisfactory stability against sliding, his options are usually limited to increasing the resisting forces. This can be accomplished by a vertical keyway into the foundation, a greater depth of soil at the downstream toe of the structure, or an increase in the structure's weight. Anchors constructed in underlying soil or rock will also provide resistance to sliding. A structure's weight may sometimes be increased by increasing the lateral extent of footings, which would mobilize additional earth loading to increase the value of W. If the assumption of erosion at the structure's toe results in the loss of too great a resisting force, it could be beneficial to provide more

extensive erosion protection to insure that the resisting soil remains in place.

If extensive uplift from groundwater forces reduces W substantially, the exterior drainage and seepage conditions can be modified to some degree. Filter materials, crushed rock, or PVC liners can be installed outside of a structure to convey groundwater away from the structure more quickly than it can move through the soil to the structure. Uplift pressures due to seepage can be reduced to some degree by an increase in the seepage path under the structure by use of cut-off walls or curtains. As noted in Section V. D., Lane's Weighted Creep procedure can be used for this analysis.

Inspection of Equations (14-6) and (14-7) indicates that greater benefit will be derived from increasing resisting forces acting on the structure than by increasing the downward force, *W*.

D. Uplift and Flotation

The pressures of groundwater under a hydraulic structure exert a vertical force referred to as an uplift force. The uplift force is counteracted by downward forces, which include the structure's weight, soil loads that bear downward on the structure's base or footings, friction of the soil along structure walls, and, in some cases, water within or flowing through the structure. When the uplift force exceeds the sum of the downward forces, flotation of either an entire structure or a part of a structure can occur. Figure 14.3 illustrates forces on a typical energy dissipator in an uplift evaluation (after the passage of a design flow), and on an outlet structure in a detention pond.

Flotation failures have occurred in several types of drainage structures. Pipes constructed of lightweight materials, such as corrugated steel or PVC, which extend into flood control ponds or reservoirs, can become plugged with debris to the extent that little water is in the pipe while the water continues to exert an uplift force. Lengths of pipe have been forced out of the ground under these conditions. Similarly, structures designed to be normally filled with a fluid have floated upward when they were drained for cleaning or repairs. While some flotation failures have been spectacular, most result in only a relatively small vertical displacement. Even small movements, however, can cause total failure of a structure, if slabs or walls crack or buckle.

The possibility of flotation of a structure is evaluated using Equation 14-8:

$$F.S._U = \frac{F_d}{F_u} \tag{14-8}$$

where

$F.S._U$ = factor of safety against uplift
F_d = the sum of the downward forces on the structure
F_u = the sum of the upward forces on the structure

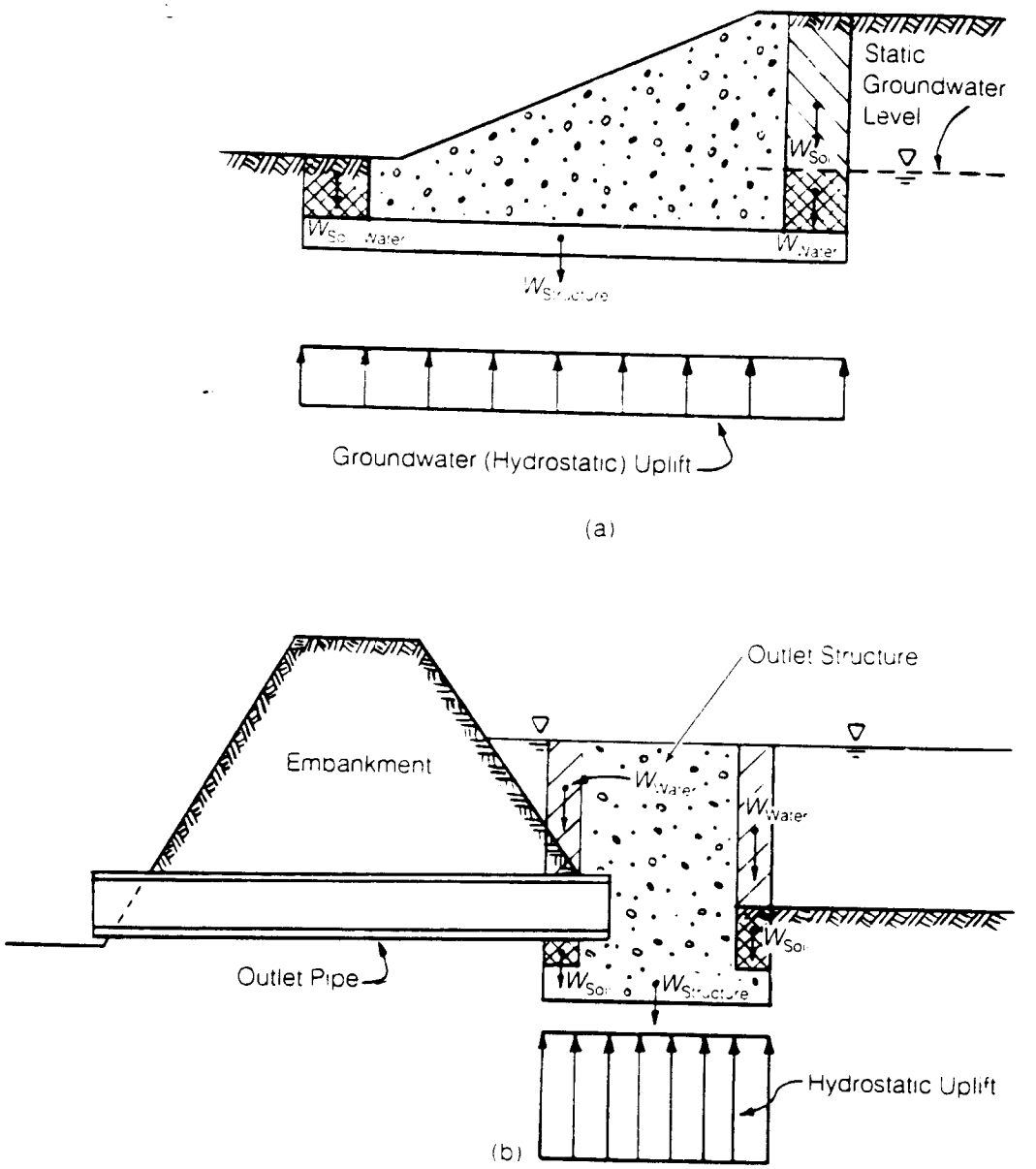


Figure 14.3—Structures subjected to uplift: (a) energy dissipator in groundwater and (b) pond outlet structure.

Design engineers have used values of $F.S._U$ as low as 1.0. However, because many of the forces are difficult to predict with accuracy, a value of $F.S._U$ of 1.25 is recommended. When a designer finds that the value of $F.S._U$ is smaller than he considers appropriate, he must generally find ways to increase F_d, such as by adding dead weight to the structure, or by extending a footing outward from the structure to mobilize additional downward soil weight, providing the footing is structurally designed to handle the loads. Uplift forces may also be resisted by anchors. In some structures it may be appropriate to include the weight of water contained in the structure, but only if its presence is assured

during events which would create the uplift. A final approach which could apply to some specific structures is a system of weep holes or similar devices which would allow groundwater to flow into and fill a structure if it reached a critical elevation. Some underground lift stations and wet wells are designed in this manner.

In some cases, F_u could be reduced by lengthening a seepage path, constructing cutoff walls or membranes, or modifying drainage under the structure. However, caution must be exercised to insure that such devices will work under all conditions.

E. Soil Bearing Pressures

Drainage or flood control structures, which have weight and may support a combination of other loads, exert a force on the underlying soil. The downward force must not exceed the capability of the soil to support the load, or excessive settlement or consolidation may occur. The downward pressure the structure may be allowed to exert on the soil is limited by two parameters. The pressure must not create a failure of the soil and must not cause settlement that would exceed the adjusting capabilities of the structure. The allowable soil bearing capacity is a parameter that must be determined by a geotechnical engineer, based on both concerns. The value is normally determined as a safe bearing capacity, and therefore includes a safety factor. As long as the downward pressure of the structure's foundation does not exceed the allowable bearing capacity, no additional safety factor is applied.

Small drainage and flood control structures seldom cause significant soil pressures by virtue of their own weight. However, some structures, such as retaining walls and dams, are designed in a way that concentrates the load (as a result of an overturning tendency) in a specific part of the footing, usually the toe. The resultant soil pressures under points of load concentration must be evaluated to insure that they will not exceed the allowable pressure. When a designer finds that the concentrated pressures exceed allowable soil capacities, he generally must increase the footing dimensions to spread the load over a larger area of the foundation.

Large drainage structures, such as dams and levees, may create loads that greatly exceed the capacity of the underlying soil. In such cases, several options exist. The load may be spread out over a larger area; the weaker underlying soil may be removed to allow the facility to bear upon deeper strata with greater capacity; or deep foundations such as piles or caissons may be employed to transfer the load to strata with adequate load-bearing capacity.

VII. DESIGN OF STRUCTURAL MEMBERS

A. General

The stability analyses described in the previous section frequently result in a refinement of the overall size of the structure. Structural

modifications to achieve stability supplement the dimensions first established to achieve the desired hydraulic function. The structural design of individual members involves the selection of the materials to be used, the sizing of the members, and the preparation of the plans and specifications.

B. Selection of Materials

The selection of the materials from which the structure will be built is frequently done prior to the structural design stage, for reasons such as aesthetics or the availability of materials. The two materials used most commonly are earth (including soil cement) and concrete (reinforced, non-reinforced, and roller-compacted). In addition, many precast or pre-formed units are used, primarily pipe. Less commonly used materials include steel sheet piling, timber, gabions, and synthetic and organic erosion protection devices.

C. Sizing of Members

Each member must be designed to withstand the loads imposed on it. In addition, each member must be connected to the adjacent members in such a way that its loads are properly transferred to the overall structure. Finally, each member must be detailed, in both contract drawings and specifications, so that it can be properly constructed.

VIII. CONDUIT STRUCTURAL REQUIREMENTS

A. Introduction

The structural design of a storm sewer requires that the supporting strength of the installed sewer pipe, divided by a suitable factor of safety, equal or exceed the loads imposed on it by the combined weight of soil and any superimposed loads.

The following are generally accepted criteria and methods for determining combined loads and supporting strength of the sewer pipe, as well as procedures for combining these elements with the application of a factor of safety to produce a safe and economical design.

Methods are presented for estimating probable maximum loads caused by soil forces and for both static and moving superimposed loads (note also that the determination of loads on rigid conduits is presented in great detail in ASCE Manual 60, *Gravity Sanitary Sewers*, Chapter 9, "Structural Requirements" (ASCE 1982)). Where so noted, the methods apply to rigid and flexible conduits in the three most common conditions of installation: in a trench in natural ground; in an embankment; and in a tunnel.

The design of rigid and flexible pipes is treated separately. There are no specific design procedures given for flexible pipes of intermediate stiffness. For such cases, design procedures such as computer analysis

based on soil-structure interaction or the designs for rigid or flexible pipes may be used (not interchangeably) for conservative results.

The supporting strength of a buried sewer pipe is a function of installation conditions as well as the strength of the sewer pipe itself. Structural analysis and design of the sewer line are problems of soil-structure interaction. This chapter presents procedures for determining the field or installed supporting strength of rigid sewer pipe based on its established relationship to the laboratory test strength, commonly called the Indirect Design Method. It also presents methods of predicting approximate field deflections for flexible pipe, based on empirical methods. Since installation conditions have such an important effect on both load and supporting strength, a satisfactory sewer construction project requires that assumed design conditions be adhered to on the job site.

This chapter does not include information on reinforced concrete design of rigid sewer pipe sections. Reference should be made to standard textbooks and to ACI/ASTM/AASHTO Specifications or FHWA or industry handbooks for such design data.

B. Loads on Sewers Caused by Gravity Earth Forces

1. General Method—Marston Theory

Marston's Theory, which is widely accepted, is used for determining the vertical load on buried conduits caused by soil forces in all of the most commonly encountered construction conditions (Marston and Anderson 1913; Marston 1930). Recent analysis and actual observation of field performance have shown that designs based on the Marston Theory yield satisfactory results, especially for small diameter conduits in narrow trenches. For larger diameter conduits, the results are conservative.

In general, the theory states that the load on a buried pipe is equal to the weight of the prism of soil directly over it, called the interior prism, plus or minus the frictional shearing forces transferred to that prism by the adjacent prisms of soil. The magnitude and direction of these frictional forces are a function of the relative settlement between the interior and adjacent soil prisms. The theory makes the following assumptions:

(a) The calculated load is the load that will develop when ultimate settlement has taken place.
(b) The magnitude of the lateral pressures that induce the shearing forces between the interior and adjacent soil prisms is computed in accordance with Rankine's theory.
(c) Cohesion is negligible except for tunnel conditions.

The general form of Marston's equation is:

$$W = CwB^2 \tag{14-9}$$

in which W is the vertical load per unit length action on the sewer pipe because of gravity soil loads; w is the unit weight of soil; B is the trench width or sewer pipe width, depending on installation conditions; and C is a dimensionless coefficient that measures the effect of the following variables:

(a) The ratio of height of fill to width of trench or sewer pipe.
(b) The shearing forces between interior and adjacent soil prisms.
(c) The direction and amount of relative settlement between interior and adjacent soil prisms for embankment conditions.

2. *Types of Loading Conditions*

Although the general form of Marston's equation includes all the factors necessary to analyze all types of installation conditions, it is convenient to classify these conditions, write a specialized form of equation, and prepare separate graphs and tables of coefficients for each.

The accepted system of classification is shown diagrammatically in Figure 14.4 and is described here briefly:

Trench conditions are defined as those in which the sewer pipe is installed in a relatively narrow trench cut in undisturbed ground and covered with soil backfill to the original ground surface.

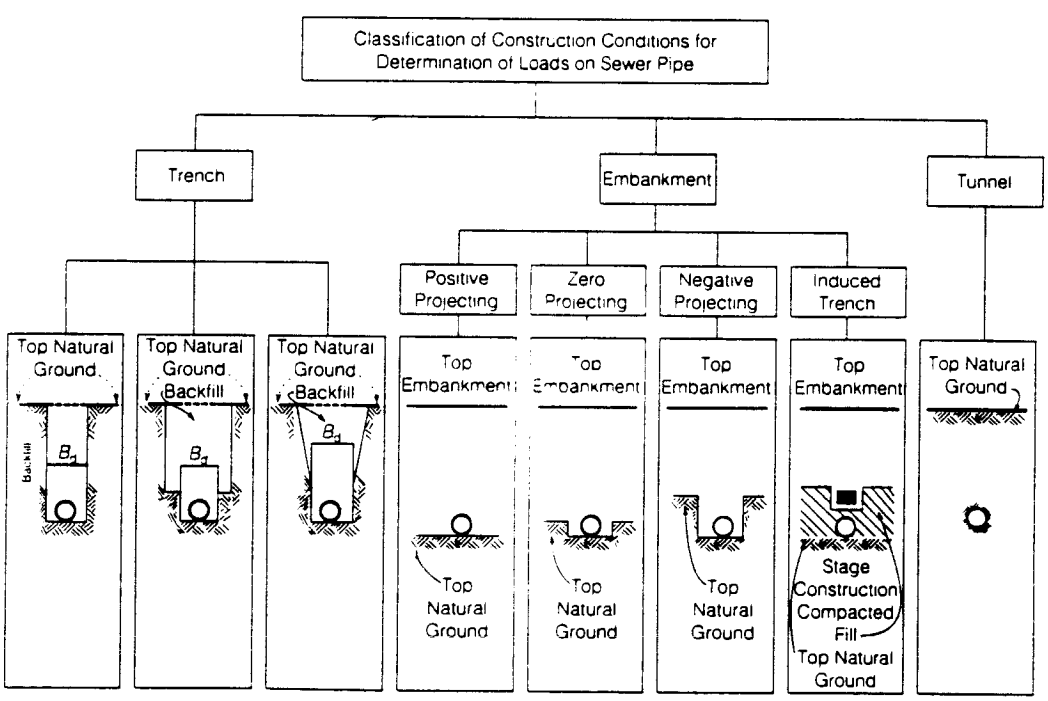


Figure 14.4—Classification of construction conditions (B_d within required limit, otherwise sewer pipe is positive projecting) (ASCE, 1969).

Embankment conditions are defined as those in which the sewer pipe is covered above the original ground surface or when a trench in undisturbed soil is so wide that trench wall friction does not affect the load on the sewer pipe. The embankment classification is further subdivided into two major subclassifications—positive projecting and negative projecting. Sewer pipe is defined as positive projecting when the top of the sewer pipe is above the adjacent original ground surface. Negative projecting sewer pipe is that installed with the top of the sewer pipe below the adjacent original ground surface in a trench that is narrow with respect to the size of pipe and the depth of cover (Figure 14.4), and when the native material is of sufficient strength that the trench shape can be maintained dependably during placement of the embankment.

A special case, called the induced trench condition, may be employed to minimize the load on a conduit under an embankment of unusual height.

3. *Loads for Trench Conditions*

Sewers usually are constructed in ditches or trenches excavated in natural or undisturbed soil, and then covered by refilling the trench to the original ground line. This procedure often is referred to as "cut and cover," "cut and fill," or "open cut."

The vertical soil load to which a sewer pipe in a trench is subjected is the result of two major forces. The first is produced by the mass of the prism of soil within the trench and above the top of the sewer pipe. The second is the friction or shearing forces generated between the prism of soil in the trench and the sides of the trench.

The backfill soil has a tendency to settle in relation to the undisturbed soil in which the trench is excavated. This downward movement or tendency for movement induces upward shearing forces that support a part of the weight of the backfill. Thus, the resultant load on the horizontal plane at the top of the sewer pipe within the trench is equal to the weight of the backfill minus these upward shearing forces (Figure 14.5) (ASCE 1969).

Unusual conditions may be encountered in which poor natural soils may effect a change from trench to embankment conditions with considerably increased load on the sewer pipe. This is covered in the next subsection.

Use of Marston's Formula Marston's formula for loads on rigid sewer pipe in trench condition is:

$$W_c = C_d w B_d^2 \tag{14-10}$$

in which W_c is the load on the sewer pipe, in pounds per foot; w is the density of backfill soil, in pounds per cubic foot; B_d is the width of trench at the top of the sewer pipe, in feet; C_d is a dimensionless load coefficient, which is a function of the ratio of height of fill to width of

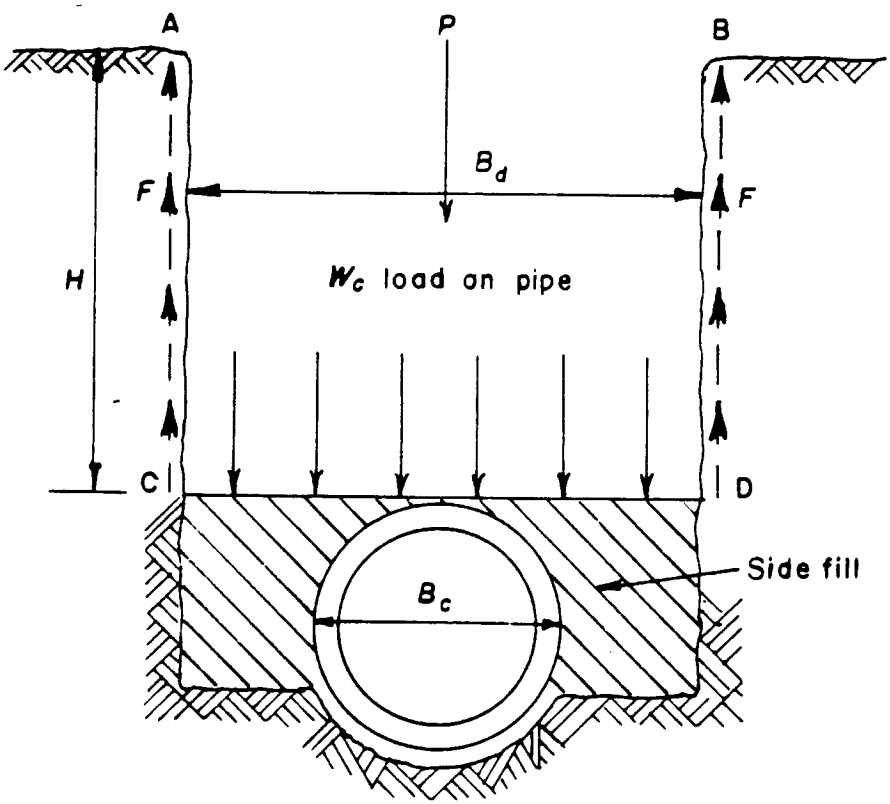


Figure 14.5—Load-producing forces: P = *weight of backfill ABCD;* F = *upward shearing forces on AC and BD, and* W_c = P − 2F *(ASCE, 1969).*

trench and of the friction coefficient between the backfill and the sides of the trench. The load coefficient, C_d, is computed as follows:

$$C_d = \frac{1 - e^{-2\kappa\mu'\frac{H}{B_d}}}{2\kappa\mu'} \tag{14-11}$$

in which e is the base of natural logarithms and κ is Rankine's ratio of lateral pressure to vertical pressure:

$$\kappa = \frac{\sqrt{\mu^2 + 1} - \mu}{\sqrt{\mu^2 + 1} + \mu} = \frac{1 - \sin\phi}{1 + \sin\phi} \tag{14-12}$$

The other terms are: $\mu = \tan\phi$ = the coefficient of internal friction of backfill material; $\mu' = \tan\phi'$ = the coefficient of friction between backfill material and sides of trench (μ' may be equal to or less than, but never greater than μ); and H is the height of fill above top of pipe, in feet. The value of C_d for various ratios of H/B_d and various types of soil backfill may be obtained from Figure 14.6.

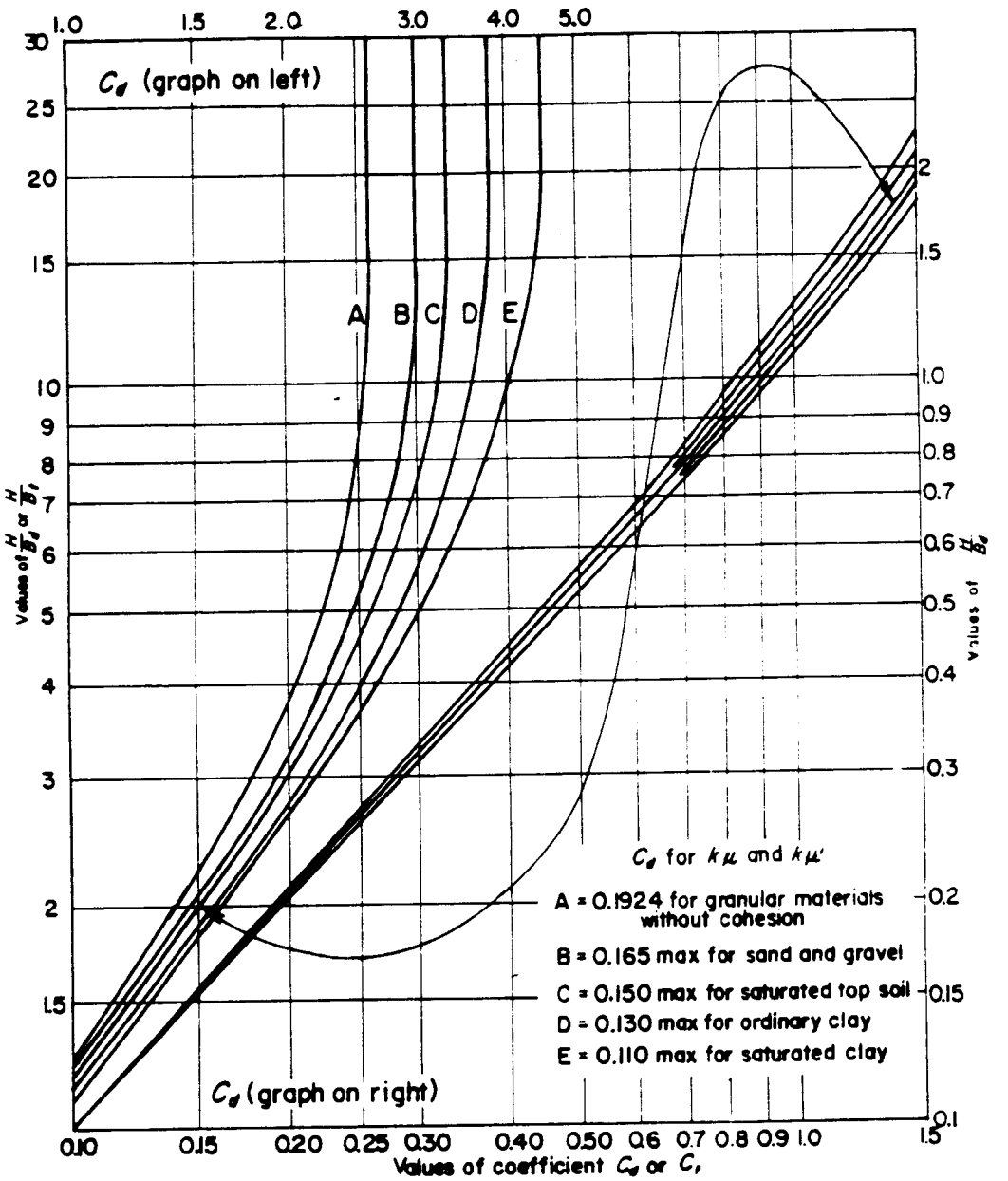


Figure 14.6—Computation diagram for soil loads on trench installations (sewer pipe completely buried in trenches) (ASCE, 1969).

The trench load formula, Equation (14-10), gives the total vertical load on a horizontal plane at the top of the sewer pipe. If the sewer pipe is rigid, it will carry practically all this load. If the sewer pipe is flexible and the soil at the sides is compacted to the extent that it will deform under vertical load less than the sewer pipe itself will deform, the side fills may carry a proportional share of the total load. Under these circumstances the trench load formula may be modified to:

$$W_c = C_d w B_c B_d \tag{14-13}$$

in which B_c is the outside width of pipe, in feet.

It is emphasized that Equation 14-13 is applicable only if the backfill is compacted as described above. The equation should not be used merely because the pipe is a flexible type.

The term "side fill" refers to the soil backfill placed between the sides of sewer pipe and the sides of the trench. The character of this material and the manner of its placement have two important influences on the structural behavior of a sewer pipe. First, the side fill may carry a part of the total vertical load on the horizontal plane at the elevation of the top of the sewer pipe. Second, the side fill plays an important role in helping the sewer pipe carry vertical load. Every pound of force that can be brought to bear against the sides of an elastic ring increases the ability of the ring to carry vertical load by nearly the same amount.

Examination of Equation 14-10 indicates the important influence the width of the trench exerts on the load as long as the trench condition formula applies. This influence has been extensively verified through experimentation. These experiments also have indicated that the width of trench at the top of the sewer pipe is the controlling factor.

The width of trench below the top of the sewer pipe also is important. It must not be permitted to exceed the safe limit for the strength of sewer pipe and class of bedding used. The minimum width must be consistent with the provision of sufficient working space at the sides of the sewer pipe to assemble joints properly, to insert and strip forms, and to compact backfill. The design engineer must allow reasonable tolerance in width for variations in field conditions and accepted construction practice.

Narrow trench construction can result in some degree of soil arching, thereby reducing the load on the pipe. Conservative design practice for pipe is based on the prism load. There are other approaches that may be considered for load calculation, some of which are discussed in later sections.

The position of the lower wale usually will determine the proper width of trench (from sheeting face to sheeting face, where sheeting and bracing are required). A working-room allowance of 12" from each side of the sewer pipe or sewer pipe cradle to the face of the sheeting is a practical minimum for small and medium-sized sewer pipe for trenches up to about 13' deep.

At any given depth and for any given sewer pipe size there is a certain limiting value to the width of trench beyond which no additional load is transmitted to the sewer pipe. This limiting value is called the "transition width." There are sufficient experimental data to show that it is safe to calculate the imposed load by means of the trench-conduit formula, Equation (14-10), for all widths of trench less than that which gives a load equal to the load calculated by the projecting-conduit formula (see section 4a). In other words, as the width of the trench

increases (other factors remaining constant), the load on a rigid sewer pipe increases in accordance with the theory for a trench sewer pipe until it equals the load determined by the theory for a projecting sewer pipe. The width of trench at which this transition occurs may be determined from Figure 14.7 (Schlick 1932). The curves in Figure 14.7 are calculated for sand and gravel, where $\kappa\mu = 0.165$, but can be used for other types of soil since the change with varying, values of $\kappa\mu$ is small. In any event, the design engineer can check by calculating loads for both trench and embankment conditions. There is little research on the appropriate value of $r_{sd}p$ (the projection ratio times the settlement ratio) to use in the application of the transition width concept. In the absence

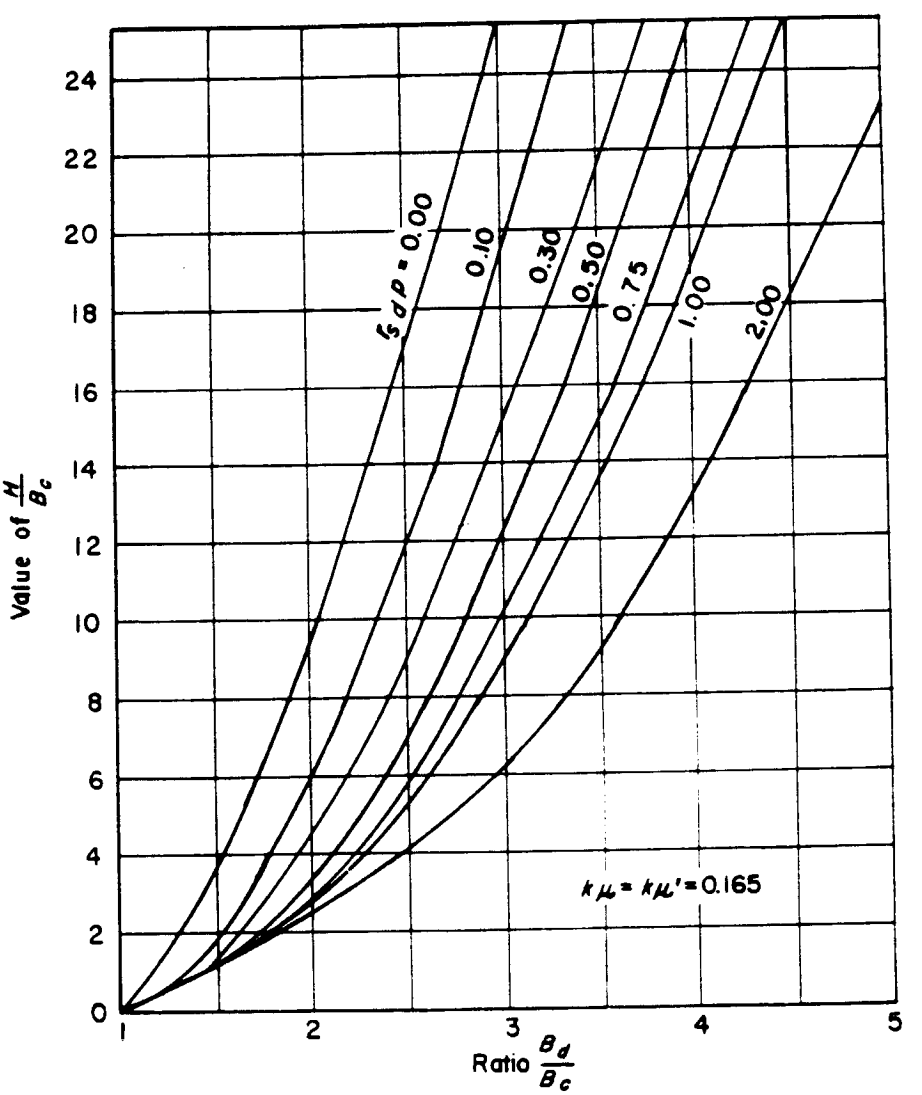


Figure 14.7—Values of B_d/B_c at which pipe in trench and projecting pipe load formulas give equal loads (Schlick, 1932).

of specific information, a value of 0.5 is suggested as a reasonably good working value. These quantities are defined in section 4.

It is advisable, in the structural design of sewers, to evaluate the effect of the transition width on both the design criteria and the construction procedure. A contractor, for instance, may wish to place well points for drainage in the trench. If this requires a wider trench than usual, a stronger sewer pipe or higher class of bedding may be necessary.

It may be economical and proper to excavate the trench with sloping sides in undeveloped areas where no inconvenience to the public or danger to property, buildings, subsurface structures, or pavements will result. A subtrench (Figure 14.8) may be used in such cases to minimize the load on the pipe. When sheeting of the subtrench at the pipe is necessary, it should extend about 1.5′ above the top of the pipe.

When sheeting of the trench is necessary, it should be driven at least to the bottom of the pipe bedding or foundation material, if used. In general, in a constantly wet or dry area, sheeting and bracing should be left in place to prevent reduction in lateral support at the sides of the pipe because of voids formed by removal of the sheeting. Sheeting left in place should be cut off as far below the surface as practicable, but in no case less than 3 feet below final ground elevation.

When wood sheeting is to be removed, the sheeting should be cut off 1.5 feet above the top of the pipe and the sheeting alongside the pipe left in place. Steel sheeting to be removed should be pulled in increments as the trench is backfilled, and the soil should be compacted to prevent formation of voids. The portion of wood sheeting to be removed should be handled similarly.

Loads on sewer pipe in sheeted trenches should be calculated from a trench width measured to the outside of the sheeting if it is pulled or to the inside if it is left in place. Voids created by removal of the sheeting should be backfilled with a flowable material such as pea gravel.

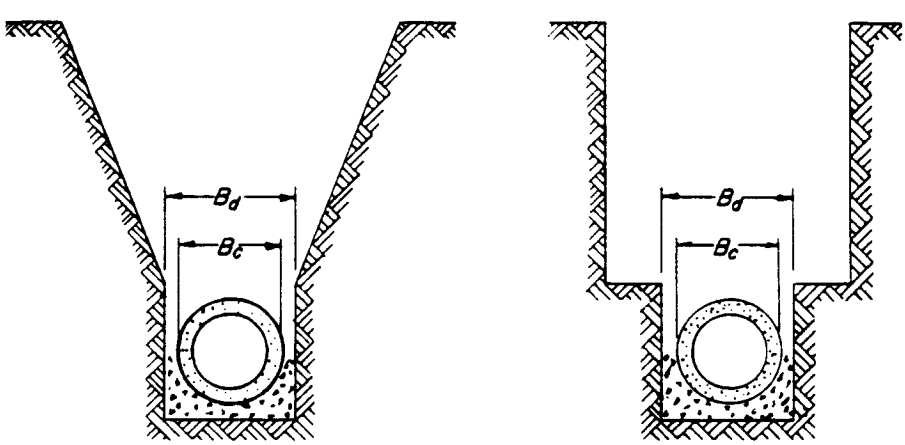


Figure 14.8—Examples of subtrench (ASCE, 1969).

If a shield is used in sewer pipe-laying operations, the shield width controls the width of the trench at the top of the sewer pipe. This width, with a small addition for the space needed to advance the shield without a large friction loss, should be the width factor used in computing loads on the sewer pipe. Extreme care must be taken when advancing the shield in the trench to prevent the pipe joints from pulling apart or to avoid disturbance of the pipe bedding.

Sewers to be constructed in sloping-sided trenches with the slopes extending to the invert, or to any place above the invert but below the top of the sewer, should be designed for loads computed by using the actual width of the trench at the top of the sewer pipe, or by the projecting-sewer formula, whichever gives the largest load on the sewer pipe.

If for any reason the trench becomes wider than that specified and for which the sewer pipe was designed, the load on the sewer pipe should be checked and a stronger sewer pipe or higher class of bedding used if necessary.

Soil Characteristics The load on a sewer pipe is influenced directly by the density of the soil backfill. This value varies widely for different soils, from a minimum of about 100 pounds per cubic foot to a maximum of about 135 pounds per cubic foot. The average maximum unit weight of the soil that will constitute the backfill over the sewer pipe may be determined by density measurements in advance of the structural design of the sewer pipe. A design value of not less than about 125 pounds per cubic foot is recommended if such measurements are not made.

The load is also influenced by the coefficient of friction between the backfill and the sides of the trench, and by the coefficient of internal friction of the backfill soil. Ordinarily these two values will be nearly the same and may be so considered for design purposes, as in Figure 14.6. However, in special cases this may not be true. For example, if the backfill is sharp sand and the sides of the trench are sheeted with finished lumber, μ may be substantially greater than μ'. Unless specific information to the contrary is available, values of the products $\kappa\mu$ and $\kappa\mu'$ may be assumed to be the same and equal to 0.130. If the backfill soil is a "slippery" clay and there is a possibility that it will become very wet after being placed, $\kappa\mu$ and $\kappa\mu'$ equal to 0.110 (maximum for saturated clay, Figure 14.6) should be used.

4. *Loads for Embankment Conditions*

A sewer pipe is described as a projecting sewer pipe when installed in a wide trench in such a manner that the top of the sewer pipe is at or near the natural ground surface or the surface of thoroughly compacted soil and subsequently is covered with an embankment. If the top of the sewer pipe projects some distance above the natural ground surface, or if it is installed in a wide trench, it is a positive projecting sewer pipe. Other methods of installing sewer pipe under embank-

ments, however, have the favorable effect of minimizing the load on the pipe. In these cases, the installation is classified as a negative projecting sewer pipe or an induced trench sewer pipe (Figure 14.4).

Positive Projecting Sewer Pipe The load on a positive projecting sewer pipe is equal to the weight of the prism of soil directly above the structure, plus (or minus) vertical shearing forces that act on vertical planes extending upward into the embankment from the sides of the sewer pipe. For an embankment installation of sufficient height, these vertical shearing forces may not extend to the top of the embankment, but terminate in a horizontal plane at some elevation above the top of the sewer pipe known as the "plane of equal settlement" (see Figure 14.9). The shear increment acts downward when $(s_m + s_g) > (s_f + d_c)$, and upward when $(s_f + d_c) > (s_m + s_g)$, where s_m is the compression of the columns of soil of height pB_c; s_g is the settlement of the natural ground adjacent to the sewer pipe; s_f is the settlement of the bottom of the sewer pipe; and d_c is the deflection of the sewer pipe.

The location of the plane of equal settlement is determined by equating the total strain in the soil above the pipe to that in the side fill plus the settlement of the critical plane. When the plane of equal settlement is an imaginary plane above the top of the embankment (i.e., shear forces extend to the top of the embankment), the installation is called

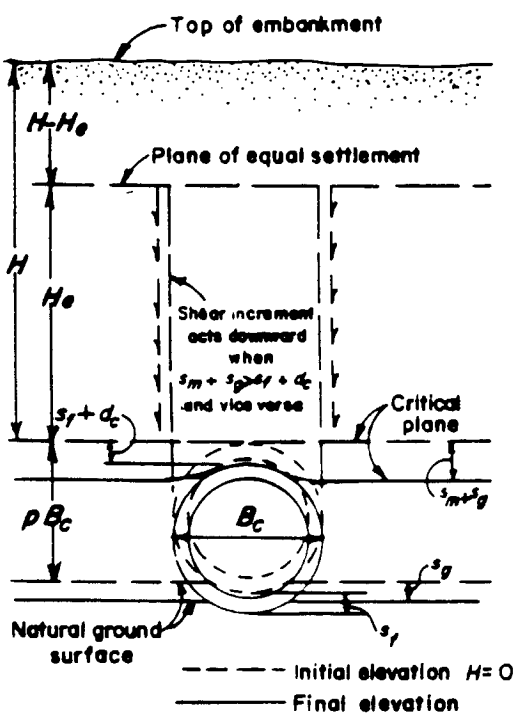


Figure 14.9—Settlements that influence loads on positive projecting sewer pipe: s_g = *settlement of natural ground adjacent to sewer pipe,* s_m = *compression of columns of soil of height* pB_c, d_c = *deflection of sewer pipe, and* s_f = *settlement of bottom of sewer pipe (ASCE, 1969).*

either "complete trench condition" or "complete projection condition" depending on the direction of the shear forces. When the plane of equal settlement is located within the embankment (Figure 14.9) (ASCE 1969), the installation is called "incomplete trench condition," or "incomplete projection condition."

In computing the settlement values, the effect of differential settlement caused by any compressible layers below the natural ground surface also must be considered. An exceptional situation for a sewer pipe in a trench can be encountered where the natural soil settles more than the trench backfill, such as where the natural soils are organic or peat, and the trench backfill is relatively incompressible compacted fill. A more common situation is where the sewer pipe is pile-supported in organic soils. In such cases, the load on the sewer pipe is greater than that of the prism above the pipe, and downward drag loads should be considered in the design of the piles.

Marston's Formula Marston's formula for loads on rigid positive projecting sewer pipe is:

$$W_c = C_c w B_c^2 \tag{14-14}$$

in which W_c is the load on the sewer pipe, in pounds per foot; B_c is the outside width of the sewer pipe, in feet; and C_c is the load coefficient. Values of C_c may be obtained from Figure 14.10. In this diagram, H is the height of fill above the top of the sewer pipe, in feet; B_c is the outside width of sewer pipe, in feet; p is the projection ratio; and r_{sd} is the settlement ratio (the last two terms are defined in the next subsection).

Influence of Environmental Factors The shear component of the total load on a sewer pipe under an embankment depends on two factors associated with the conditions under which the sewer pipe is installed. These are the projection ratio and the settlement ratio.

The projection ratio, p, is defined as the ratio of the distance that the top of the sewer pipe projects above the adjacent natural ground surface, or the top of thoroughly compacted fill, or the bottom of a wide trench, to the vertical outside height of the sewer pipe. It is a physical factor that can be determined in advanced stages of planning when the size of the sewer pipe and its elevation have been established.

The settlement ratio, r_{sd}, indicates the direction and magnitude of the relative settlements of the prism of soil directly above the sewer pipe and of the prisms of soil adjacent to it. In computing the settlement, the influence of any compressible layers below the sewer pipe also must be considered.

These relative settlements generate the shearing forces that combine algebraically with the weight of the central prism of soil to produce the resultant load on the sewer pipe. The settlement ratio is the quotient obtained by taking the difference between the settlement of the horizontal plane in the adjacent soil which was originally level with the

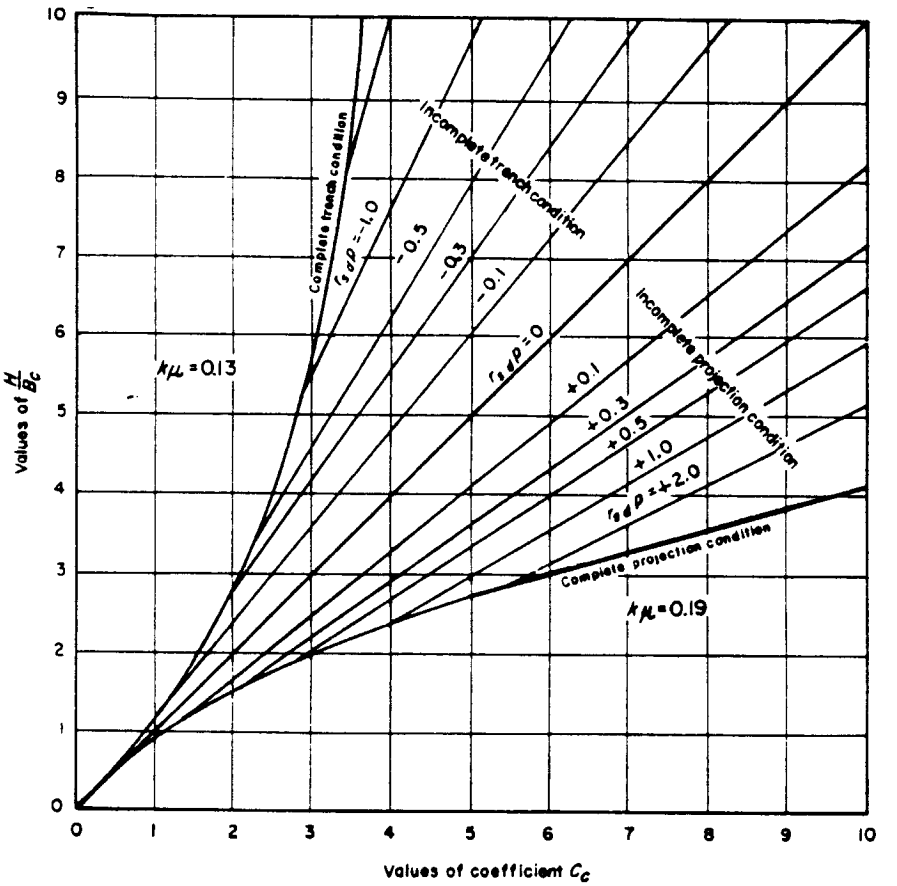


Figure 14.10—Diagram for coefficient C_c for positive projecting sewer pipes (ASCE, 1969).

top of the sewer pipe (the critical plane) and the settlement of the top of the sewer pipe, and dividing the difference by the compression of the columns of soil between the natural ground surface and the level of the top of the sewer pipe. The formula for the settlement ratio is:

$$r_{sd} = \frac{(s_m + s_g) - (s_f + d_c)}{s_m} \tag{14-15}$$

in which r_{sd} is the settlement ratio; s_g is the settlement of the natural ground adjacent to the sewer pipe; s_m is the compression of the columns of soil of height Pb_c; $(s_m + s_g)$ is the settlement of the critical plane; d_c is the deflection of the sewer pipe, that is, the shortening of its vertical dimension; s_f is the settlement of the bottom of the sewer pipe; and $(s_f + d_c)$ is the settlement of the top of the sewer pipe.

The elements of the settlement ratio are shown in Figure 14.9. When the settlement ratio is positive, the shearing forces induced along the sides of the central prism of soil are directed downward, and the load

on the sewer pipe is greater than the weight of the central prism. When the settlement ratio is negative, the shearing forces act upward and the load is less than the weight of the central prism.

The numerical magnitude of the product of the projection ratio and the settlement ratio, $r_{sd}p$, is an indicator of the relative height of the plane of equal settlement and, therefore, of the magnitude of the shear component of the load. The plane of equal settlement is at the top of the sewer pipe when this product is equal to zero. There are no induced shearing forces in this case, and the load is equal to the weight of the central prism (the "prism load").

It is not practical to predetermine a value of the settlement ratio by estimating the magnitude of its various elements except in very general terms. Rather, it should be treated as an empirical factor. Recommended design values of r_{sd}, based on measured settlements of a number of actual installations, are given in Table 14.1. The last three cases in Table 14.1 presume soil conditions immediately under the sewer pipe to be the same as those in the adjacent areas outside the trench. In these cases, the settlement ratio may be conservatively assumed as zero in locations with highly fluctuating water tables above the pipe or plastic native trench soils. This results in designing for the "prism load," or the weight of the prism of soil above the pipe. In such cases, C_c is equal to H/B_c and Marston's formula for the prism load becomes:

$$W_c = HwB_c \tag{14-16}$$

The prism load may also be expressed in terms of soil pressure, P, in pounds per square foot at depth H as:

$$P = wH = \frac{W_c}{B_c} \tag{14-17}$$

TABLE 14.1. Recommended Design Values of r_{sd}.

Type of Sewer Pipe (1)	Soil Conditions (2)	Settlement Ratio, r_{sd} (3)
Rigid	Rock or unyielding foundation	+1.0
Rigid	Ordinary foundation	+0.5 to +0.8
Rigid	Yielding foundation	0 to +0.5
Rigid	Negative projecting installation	−0.3 to −0.5
Flexible	Poorly compacted side fills	−0.4 to 0
Flexible	Well compacted side fills	0

(ASCE 1969).

Embankment Soil Characteristics The load on a projecting sewer pipe is influenced directly by the density of the embankment soil. If the soil is to be compacted to a specified dry density, the corresponding wet density under normal moisture conditions should be used in calculating the load. A design value of not less than about 125 pounds per cubic foot is recommended if specific information relative to soil density is not available.

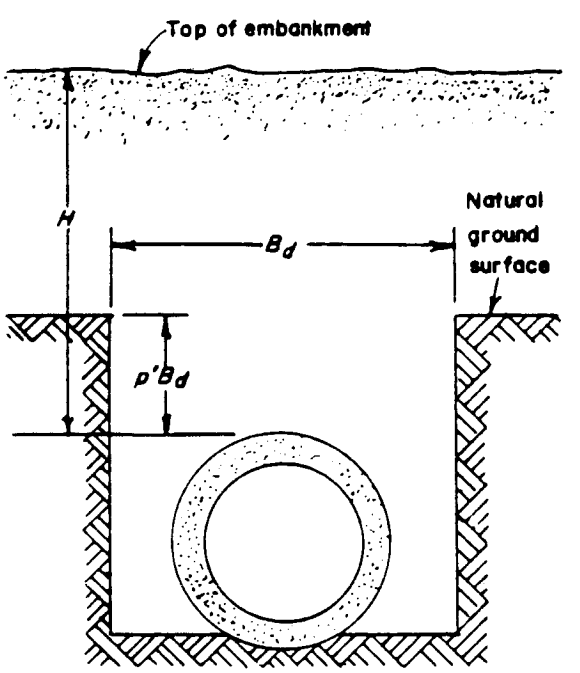
The load also is influenced by the coefficient of internal friction of the embankment soil. Recommended values of the product (Figure 14.10) are:

$$\text{for a positive settlement ratio, } \kappa\mu = 0.19$$

$$\text{for a negative settlement ratio, } \kappa\mu = 0.13$$

Negative Projecting and Induced Trench Sewer Pipes A negative projecting sewer pipe (Figure 14.11) is one installed in a relatively shallow trench with its top at some elevation below the natural ground surface.

The trench above the sewer pipe is refilled with loose, compressible material, and the embankment is constructed to finished grade by ordinary methods. The greater the value of the negative projection ratio, p', and the more compressible the trench backfill over the sewer pipe, the greater will be the settlement of the interior prism of soil in relation to the adjacent fill material. In using this technique, the plane of equal settlement must fall below the top of the finished embankment. This



Negative projecting

Figure 14.11—Negative projecting sewer pipe (ASCE, 1969).

action generates upward shearing forces that relieve the load on the sewer pipe.

An induced trench sewer pipe (Figure 14.12) first is installed as a positive projecting sewer pipe. The embankment then is built up to some height above the top and thoroughly compacted as it is placed. A trench of the same width as the sewer pipe next is excavated directly over the sewer pipe down to or near the top of the sewer pipe. This trench is refilled with loose, compressible material, and the balance of the embankment is completed in a normal manner. Sometimes straw, hay, cornstalks, sawdust or similar materials may be used in the trench backfill to augment the settlement of the interior prism.

The formula for loads on negative projecting sewer pipe is:

$$W_c = C_n w B_d^2 \tag{14-18}$$

in which W_c is the load on the sewer pipe, in pounds per foot; w is the density of soil; B_d is the width of the trench; C_n is the load coefficient (Figure 14.13) (ASCE 1969), a function of H/B_d or H/B_c, p', and r_{sd}; p' is the projection ratio; and r_{sd} is the settlement ratio as defined below.

In the case of the induced trench sewer pipe, B_c is substituted for B_d in Equation (14.18), in which B_c is the width of the sewer pipe, assuming the trench in the fill is no wider than the sewer pipe.

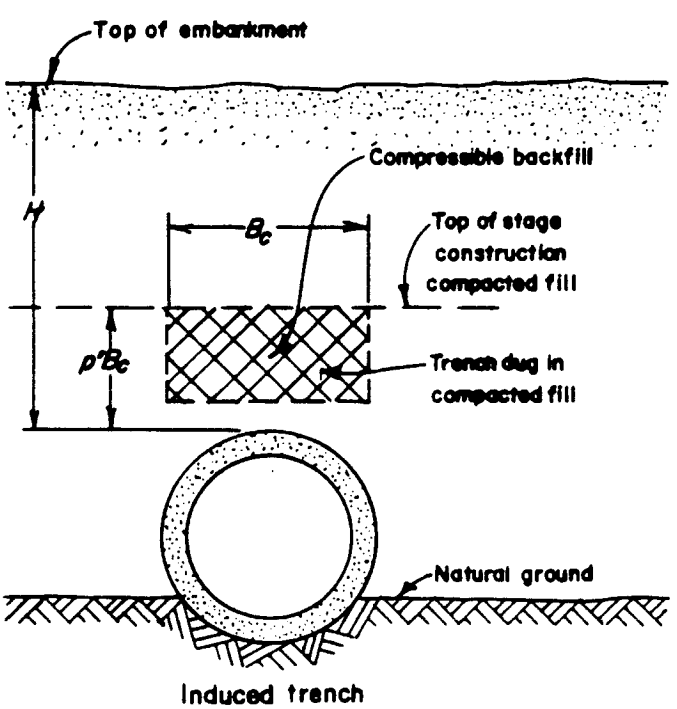


Figure 14.12—Induced trench pipe (ASCE, 1969).

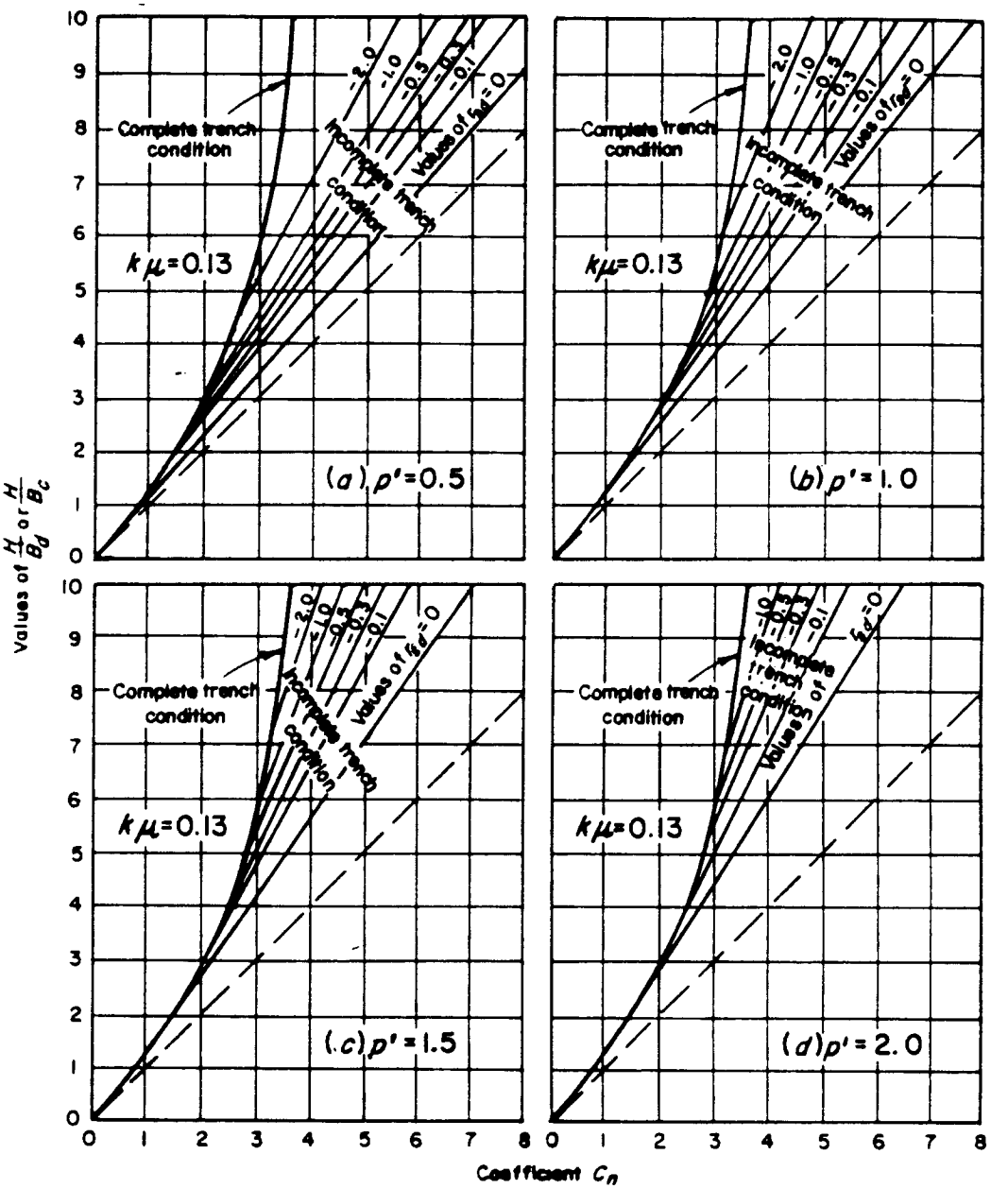


Figure 14.13—Diagrams for coefficient C_n for negative projecting and induced trench sewer pipes (ASCE, 1969).

The projection ratio, p', is equal to the vertical distance from the firm ground surface down to the top of the sewer pipe divided by the width of the trench, B_d in the case of negative projecting sewer pipe, or by the width of the sewer pipe, B_c, in the case of induced trench sewer pipe.

The settlement ratio, r_{sd}, for these cases is the quotient obtained by taking the difference between the settlement of the firm ground surface

and the settlement of the plane in the trench backfill that was originally level with the ground surface (the critical plane), and dividing the difference by the compression of the column of soil in the trench. The formula for the settlement ratio is:

$$r_{sd} = \frac{s_g - (s_d + s_f + d_c)}{s_d} \tag{14-19}$$

in which r_{sd} is the settlement ratio for negative projecting or induced trench sewer pipe; s_g is the settlement of the firm ground surface; s_d is the compression of trench backfill within the height $p'B_d$ or $p'B_c$; s_f is the settlement of the bottom of the sewer pipe; d_c is the deflection of the sewer pipe, that is, the shortening of its vertical dimension; and $(s_d + s_f + d_c)$ is the settlement of the critical plane. The elements of the settlement ratio are shown in Figure 14.14.

In the absence of extensive data on the probable values of the settlement ratio, it is tentatively recommended that this ratio be assumed to be between -0.3 and -0.5. Research (Taylor 1971) has indicated that the measured settlement ratio of 48-inch reinforced concrete pipe

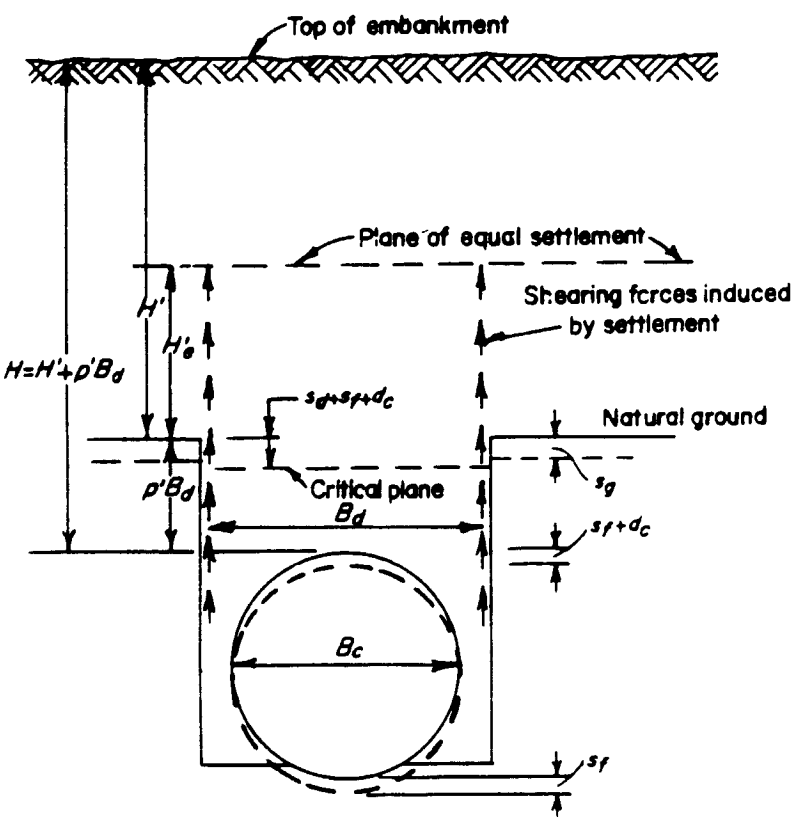


Figure 14.14—Settlements that influence loads on negative projecting sewer pipes (ASCE, 1969).

culvert, installed as an induced trench conduit under approximately 30′ of fill, varied from −0.25 to −0.45.

Sewer Pipe Under Sloping Embankment Surfaces Sewer pipe can sometimes have different heights of fill on two sides because of the sloping surface of the embankment or when embankment exists on one side of the sewer pipe only. Such cases require special analysis. Design based on the larger fill height may not yield conservative results. Where yielding ground may surround the sewer pipe, a surcharge on one side of the sewer pipe may result in vertical displacement.

5. *Loads for Jacked Sewer Pipe and Certain Tunnel Conditions*

When the sewer is more than 30–40′ deep or when surface obstructions are such that it is difficult to construct the sewer by trenching, it may be more economical to place the sewer by means of jacking or tunneling. The theories set forth in this Manual usually will be appropriate for materials where jacking of the sewer pipe is possible and for tunnels in homogeneous soils of low plasticity. Where a tunnel is to be constructed through materials subject to unusually high internal pressures and stresses, such as some types of clays or shales that tend to squeeze or swell, or through fissured and seamed rock, the loads on the sewer pipe cannot be determined from factors discussed here. Reference should be made to the following section on tunnels.

The methods of constructing sewers by tunneling and jacking are described in Chapter 16. Tunnel supports carry the earth load until the sewer pipe is constructed and the voids between the sewer pipe and tunnel supports are filled. Jacked sewer pipe is assumed to carry the earth load as it is pushed into place (Contractors and Engineers Monthly 1948; ACPA 1960).

Load-Producing Forces The vertical load acting on the jacked sewer pipe or tunnel supports, and eventually the sewer pipe in the tunnel, is the result of two major forces; the weight of the overhead prism of soil within the width of the jacked sewer pipe or tunnel excavation and the shearing forces generated between the interior prisms and the adjacent material.

During excavation of a tunnel, and varying somewhat with construction method, the soil directly above the face of the tunnel tends to settle slightly in relation to the soil adjacent to the tunnel because of the lack of support during the period immediately after excavation and prior to placement of the tunnel support. Also, the tunnel supports and the sewer pipe must deflect and settle slightly when the vertical load is applied. This downward movement or tendency for movement induces upward shearing forces that support a part of the weight of the earth prism above the tunnel. The cohesion of the material provides additional support. The resultant load on the horizontal plane on the top of the tunnel and within the width of the tunnel excavation is equal

to the weight of the prism of earth above the tunnel minus the upward friction forces and cohesion of the soil along the limits of the prism of soil over the tunnel.

Hence, the forces involved with gravity earth loads on jacked sewer pipe or tunnels in such soils are similar to those discussed for loads on sewer pipe in trenches except for the cohesion of the material. Cohesion also exists in the case of loads in trenches and embankments, but is neglected because the cohesion of the disturbed soil is of minor consequence and may be absent altogether if the soil is saturated. However, in the case of jacked sewer pipe, or in tunnels where the soil is undisturbed, cohesion can reduce loads appreciably, and may be considered safe if reasonable coefficients are assumed.

Jacking stresses must be investigated in pipe that is to be jacked into place. The critical section is at the pipe joint where the transfer of stress from one pipe to the adjacent pipe occurs. Jointing materials should be used that will provide uniform bearing around the pipe circumference (ACPA 1960). Thrust at the joint is usually transmitted through the tongue or groove but not both. Concrete stress in the tongue or groove should be checked and additional reinforcement for both longitudinal and bursting stresses provided if required.

Marston's Formula When modified to include cohesion, Marston's formula may be used to determine the gravity soil loads on jacked sewer pipe or sewer pipe in tunnels through undisturbed soil (Figure 14.15). The modified Marston formula is as follows:

$$W_t = C_t B_t (wB_t - 2c) \tag{14-20}$$

in which W_t is the load on the sewer pipe or tunnel support, in pounds per foot; w is the density of the soil above the tunnel; B_t is the maximum width of the tunnel excavation (B_c in the case of jacked sewer pipe); c is the cohesion coefficient, in pounds per square foot; and C_t is a load coefficient, which is a function of the ratio of the distance from the ground surface to the top of the tunnel to the width of the tunnel excavation and of the coefficient of internal friction of the natural material above the tunnel.

The formula for C_t is identical to that for C_d (Equation 14-11), except that H is the distance from the ground surface to the top of the tunnel and B_t is substituted for B_d. The values of the coefficient for C_t for various ratios of H/B_t and various types of materials may be obtained from Figure 14.16 (ASCE 1969) or Figure 14.6. Values of $\kappa\mu$ and $\kappa\mu'$ are the same as those noted in Figure 14.6.

An analysis of the formula for computing C_t indicates that for very high values of H/B_t the coefficient C_t approaches the limiting value of $1/(2\kappa\mu')$. Hence, where the tunnel is very deep, the load on the tunnel can be calculated readily by using the limiting value of C_t.

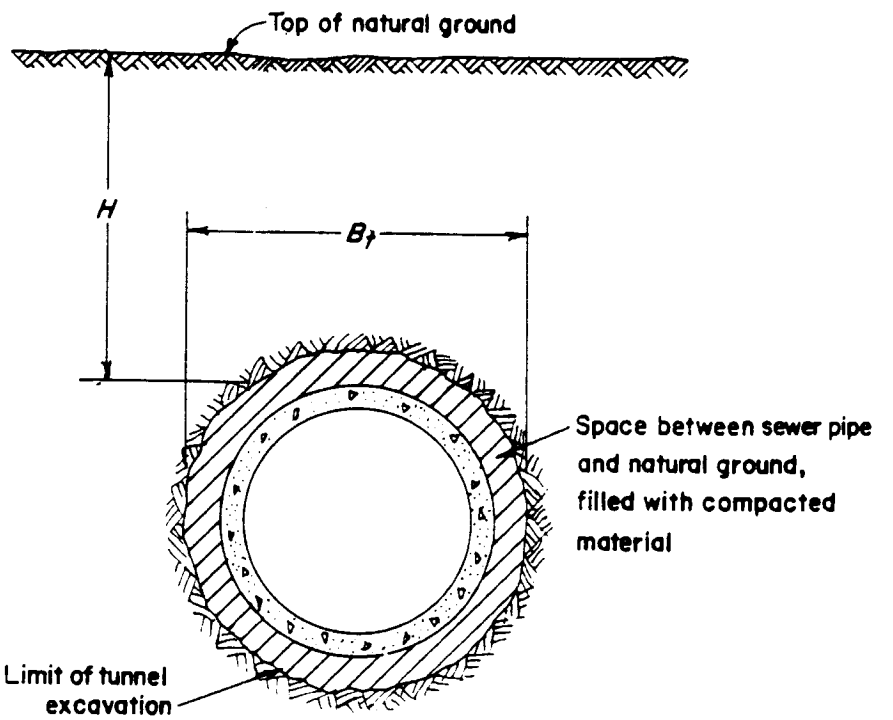


Figure 14.15—Sewer pipe in tunnel (ASCE, 1969).

Tunnel Soil Characteristics The discussion regarding unit weight and coefficient of friction for sewers in trenches applies equally to the determination of earth loads on jacked sewer pipe or sewer pipe in tunnels through undisturbed soil.

The one additional factor that enters into the determination of loads on tunnels is c, the coefficient of cohesion. An examination of Equation (14-20) shows that the proper selection of c is very important; unfortunately, it can vary widely even for similar types of soils. It may be possible in some instances to obtain undisturbed samples of the material and to determine the value of c in the laboratory. Such testing should be done whenever possible. Conservative values of c should be used to allow for a saturated condition of the soil or for other unknown factors. Design values should probably be about 33% of the laboratory test value to allow for uncertainties.

For cases in which it is not practicable to determine c (the coefficient of cohesion) from laboratory tests, recommended safe values of c are listed in Table 14.2.

It is suggested that the value of c be taken as zero in the zone subject to seasonal frost and cracking because of dessication or loss of strength from saturation.

Effect of Excessive Excavation Where the tunnel is constructed by a method that results in excessive excavation and where the voids above

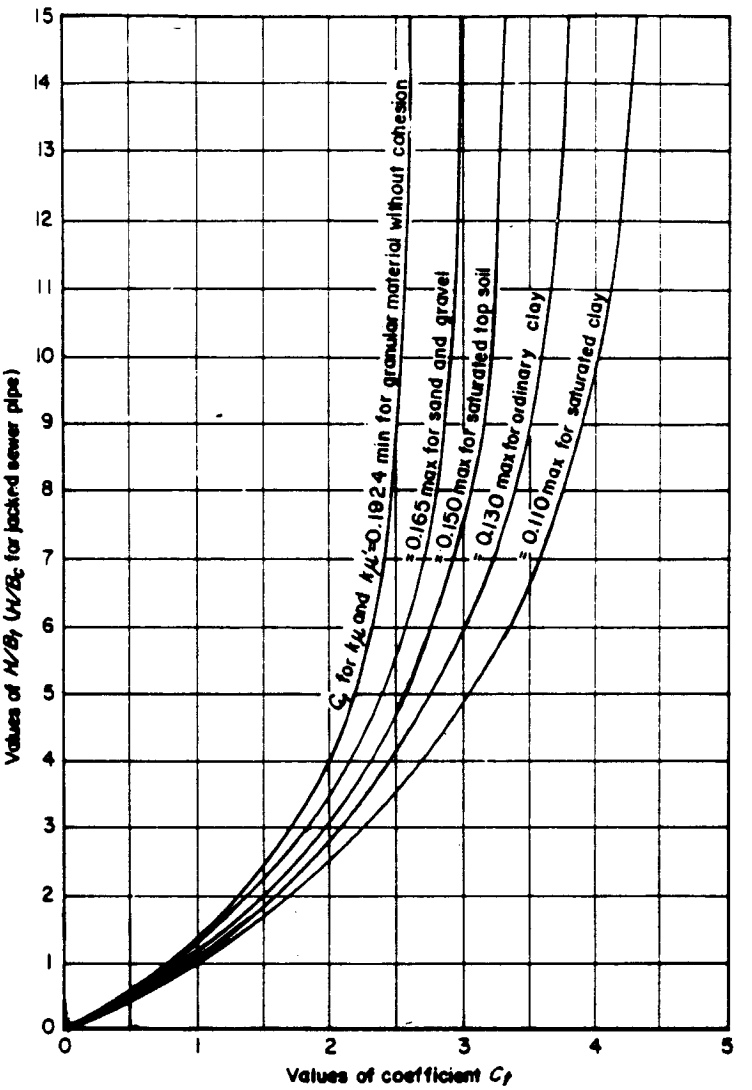


Figure 14.16—Diagram for coefficient C_t for jacked sewer pipe or tunnels in undisturbed soil (ASCE, 1969).

TABLE 14.2. Recommended Safe Values of Cohesion, *c*.

Material (1)	Values of *c*	
	In Kilopascals (2)	In Pounds per Square Foot (3)
Clay, very soft	2	40
Clay, medium	12	250
Clay, hard	50	1,000
Sand, loose dry	0	0
Sand, silty	5	100
Sand, dense	15	300

(ASCE 1969).

the sewer pipe or tunnel lining are not backfilled carefully or packed with grout or other suitable backfill materials, saturation of the soil or vibration eventually may destroy the cohesion of the undisturbed material above the sewer pipe and result in loads in excess of those calculated using Equation (14-20). If this situation is anticipated, it is suggested that Equation (14-20) be modified by eliminating the cohesion term. The calculated loads then will be the same as those calculated for the trench condition from Equation (14-10).

6. *Loads for Tunnels*

When the sewer is to be constructed in a tunnel through homogeneous soils of low plasticity, the design should be based on the theories set forth in the previous section describing jacked sewer pipe. The design of tunnels through other types of materials is discussed in this section. The usual procedure in tunnel construction is to complete the excavation first, then place either a cast-in-place concrete liner or sewer pipe, and then grout or concrete the pipe in place. Additional strength in such a section can be obtained by means of pressure grouting to strengthen the surrounding material instead of relying totally on the liner or pipe itself. Tunnel loads, therefore, usually are determined for purposes of selecting supports to be used during excavation, and the sewer pipe or cast-in-place liner is designed primarily to withstand loads from pressure grouting.

Load-Producing Forces When the tunnel is to be constructed through soils that tend to squeeze or swell (such as some types of clay or shale), or through fissured or seamed rock, the vertical load cannot be determined from a consideration of the factors discussed previously, and Equation (14-20) is not applicable.

The determination of rock pressures exerted against the tunnel lining is largely an estimate based on previous experience of the performance of linings in similar rock formations, although attempts at numerical analysis of stress conditions around a tunnel shaft have been made.

In the case of plastic clay, the full weight of the overburden is likely to come to rest on the tunnel lining some time after construction. The extent of lateral pressures to expect is yet to be determined fully, especially the passive resistance which will be maintained permanently by a plastic clay in the case of a flexible ring-shaped tunnel lining. For normally consolidated clays, suggested lateral pressures are on the order of $\frac{2}{3}$ to $\frac{7}{8}$ of vertical overburden pressures.

On the other hand, when tunneling through sand, only part of the weight of the overburden will come to rest on the tunnel lining at any time if adequate precautions are taken. The relief will be the result of the transfer of the soil weight immediately above the tunnel to the adjoining soil mass by shearing stresses along the vertical planes. In this case Marston's formula may be used for estimating the total load the tunnel lining may have to carry.

Great care must be taken to prevent any escape of sand into the tunnel during construction. Moist sand usually will arch over small openings and not cause trouble in this respect; however, entirely dry sand, which is sometimes encountered, is liable to trickle into the tunnel through gaps in the temporary lining. Wet sand or sand under the natural water table will flow readily through the smallest gaps. Sand movements of this kind destroy most if not all of the arching around the tunnel, resulting in a significant increase in both vertical and horizontal pressures on the supports of the lining. Such cases have been recorded and have caused considerable difficulty.

7. *Alternate Design Method*

For large diameter sewers, such as those greater than 48 inches in diameter, Indirect Designs based on the Marston method may yield conservative results. In such cases a more precise analysis can be made using a Direct Design method based on the principles of soil-structure interaction. Analysis should consider both the geometry of the system and material properties of the sewer pipe and the surrounding soil mass.

A simpler method based on arch analysis, which considers only the geometry of the sewer pipe and section properties of the sewer pipe material (Portland Cement Association 1975), can also be used for any specified loading condition.

In the method of soil-structure interaction analysis, loadings on the sewer pipe are automatically generated from the specified boundary conditions, the material properties, and the constitutive relationships of material behavior. Most solutions consider elastic behavior of the materials. Elastoplastic behavior and nonlinear analyses are also available.

The arch analysis method requires specification of vertical and lateral loads. The vertical loads can be determined by the Marston method and distributed uniformly over the full width of the sewer pipe. Lateral loads depend on the soil type and geologic history of the soil deposit. Design parameters should be obtained from a soils consultant knowledgeable in the subsurface conditions in the area. For sewer pipe installed in a tunnel or in a trench with properly compacted backfill, the recommended design lateral pressures are those corresponding to "at-rest" conditions. Where the backfill on the sides of the sewer may be loosely placed or insufficiently compacted, "active" pressure coefficients should be used to determine the lateral pressures. For preliminary analysis, the "at-rest" pressure coefficients in Table 14.3 are suggested.

Since active and passive earth pressures are the result of lateral strain in the soil mass, the at-rest condition refers to the lateral pressures existing in a large soil mass not subject to horizontal forces or strains except those resulting from its own weight.

TABLE 14.3. "At-Rest" Pressure Coefficients.

Soil Type (1)	"At-Rest" Coefficient (2)
Granular soils	0.5 to 0.67
Cohesive soils, medium to hard	0.67 to 0.88
Cohesive soils, soft	0.75 to 1.0

C. SUPERIMPOSED LOADS ON SEWERS

1. General Method

Two types of superimposed loads are encountered commonly in the structural design of sewers; concentrated and distributed. Loads on sewer pipe caused by these superimposed loads can be determined by application of Boussinesq's solution for stresses in semi-infinite elastic medium (Spangler and Hennessy 1946).

Other methods, like the one given in the AASHTO Code, can be used to determine loads on sewer pipe from superimposed loads (AASHTO undated). The AASHTO method is intended for use with wheel loads, and may not be conservative or applicable for other types of loads, such as those from adjacent building foundations. Empirical studies indicate the difficulties of accurately predicting the actual loads on the pipe.

In the design of buried sewer pipe systems, proper consideration of construction loads is necessary. Loads resulting from heavy equipment and reduced backfill heights can produce loads on the sewer pipe that exceed final design loads.

2. *Boussinesq Solution*

Concentrated Loads The formula for load caused by a superimposed concentrated load (Figure 14.17), is as follows:

$$W_{sc} = C_s \frac{PF}{L} \tag{14-21}$$

in which W_{sc} is the load on the sewer pipe, in pounds per unit length; P is the concentrated load, in pounds; F is the impact factor; C_s is the load coefficient (Table 14.4), a function of $B_c/2H$ and $L/2H$; H is the height of fill from the top of sewer pipe to ground surface; B_c is the width of sewer pipe; and L is the effective length of sewer pipe.

The effective length of a sewer pipe is defined as the length over which the average load caused by surface load produces nearly the

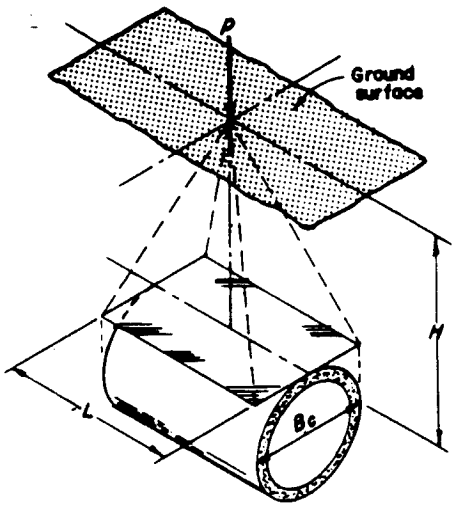


Figure 14.17—Concentrated superimpopsed load vertically centered over sewer pipe (ASCE, 1969).

same stress in the sewer pipe wall as does the actual load, which varies in intensity from point to point. Little information is available on this subject.

If the concentrated load is displaced laterally and longitudinally from a vertically centered location over the section of sewer pipe under construction, the load on the pipe can be computed by adding algebraically the effect of the concentrated load on various rectangles each with a corner centered under the concentrated load. Values of C_s in Table 14.4 divided by 4 equal the load coefficient for a rectangle the corner of which is vertically centered under the concentrated load.

Impact Factor The impact factor, F, reflects the influence of dynamic loads caused by traffic at the ground surface. Suggested values for various kinds of traffic are shown in Table 14.5.

The impact effect decreases with increasing cover. The AASHTO (highway) Code (AASHTO undated) recommends a reduction to 1.0 where depth of cover exceeds 3′ or the pipe outside diameter, whichever is larger. The AREA (railway) Code (AREA 1981–1982) recommends 10 feet of cover for the elimination of impact effect.

Distributed Loads For the case of a superimposed load distributed over an area of considerable extent (Figure 14.18), the formula for load on the sewer pipe is:

$$W_{sd} = C_s p F B_c \tag{14-22}$$

in which W_{sd} is the load on the sewer pipe, in pounds per unit length; p is the intensity of distributed load, in pounds per square foot; F is the impact factor; B_c is the width of the sewer pipe; and C_s is the load

TABLE 14.4. Values of Load Coefficients, C_S, for Concentrated and Distributed Superimposed Loads Vertically Centered over Sewer Pipe[a]

$\frac{D}{2H}$ or $\frac{B_c}{2H}$	$\frac{M}{2H}$ or $\frac{L}{2H}$													
	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.5	2.0	5.0
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
0.1	0.019	0.037	0.053	0.067	0.079	0.089	0.097	0.103	0.108	0.112	0.117	0.121	0.124	0.128
0.2	0.037	0.072	0.103	0.131	0.155	0.174	0.189	0.202	0.211	0.219	0.229	0.238	0.244	0.248
0.3	0.053	0.103	0.149	0.190	0.224	0.252	0.274	0.292	0.306	0.318	0.333	0.345	0.355	0.360
0.4	0.067	0.131	0.190	0.241	0.284	0.320	0.349	0.373	0.391	0.405	0.425	0.440	0.454	0.460
0.5	0.079	0.155	0.224	0.284	0.336	0.379	0.414	0.441	0.463	0.481	0.505	0.525	0.540	0.548
0.6	0.089	0.174	0.252	0.320	0.379	0.428	0.467	0.499	0.524	0.544	0.572	0.596	0.613	0.624
0.7	0.097	0.189	0.274	0.349	0.414	0.467	0.511	0.546	0.584	0.597	0.628	0.650	0.674	0.688
0.8	0.103	0.202	0.292	0.373	0.441	0.499	0.546	0.584	0.615	0.639	0.674	0.703	0.725	0.740
0.9	0.108	0.211	0.306	0.391	0.463	0.524	0.574	0.615	0.647	0.673	0.711	0.742	0.766	0.784
1.0	0.112	0.219	0.318	0.405	0.481	0.544	0.597	0.639	0.673	0.701	0.740	0.774	0.800	0.816
1.2	0.117	0.229	0.333	0.425	0.505	0.572	0.628	0.674	0.711	0.740	0.783	0.820	0.849	0.868
1.5	0.121	0.238	0.345	0.440	0.525	0.596	0.650	0.703	0.742	0.774	0.820	0.861	0.894	0.916
2.0	0.124	0.244	0.355	0.454	0.540	0.613	0.674	0.725	0.766	0.800	0.849	0.894	0.930	0.956

[a]Influence coefficients for solution of Holl's and Newmark's integration of the Boussinesq equation for vertical stress.

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TABLE 14.5. Suggested Values of Impact Factor, F.

Traffic Type (1)	F (2)
Highway	1.30
Railway	1.40
Airfield runways (for taxiways, consult FAA)	1.00

(ASCE 1969).

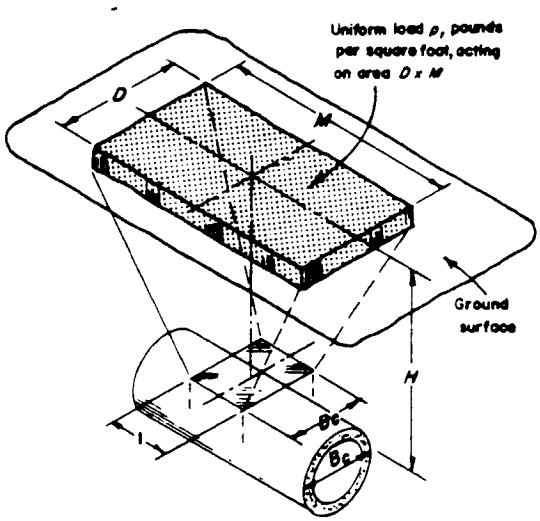


Figure 14.18—Distributed superimposed load vertically centered over sewer pipe (psf × 47.9 = Pa) (ASCE, 1969).

coefficient, a function of $D/(2H)$ and $(M/2H)$ from Table 14.4; H is the height from the top of the sewer pipe to the ground surface.

For the case of a uniform load offset from the center of the sewer pipe, the loads per unit length of the sewer pipe may be determined by a combination of rectangles. For determination of the stress below a point such as A in Figure 14.19, as a result of the loading in the rectangle BCDE, the area may be considered to consist of four rectangles: (AJDF) − (AJCG) − (AHEF) + (AHBG). Each of these four rectangles has a corner at point A. By computing $D/2H$ and $M/2H$ for each rectangle, the load coefficient for each rectangle can be taken from Table 14.4. Since point A is at the corner of each rectangle, the load coefficients from Table 14.4 should be divided by 4. A combination of the stresses from the four rectangles, with signs as indicated above, gives the desired stress.

Values of C_s can be read directly from Table 14.4 if the area of the distributed superimposed load is not centered over the sewer pipe under consideration.

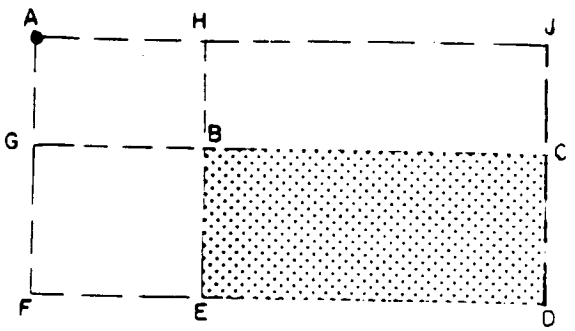


Figure 14.19—Diagram for obtaining stress at point A caused by load in shaded area BCDE (ASCE, 1969).

The load on the sewer pipe can be computed by adding algebraically the effect of various rectangles of loaded area if the area of the distributed superimposed load is not centered over the sewer pipe, but is displaced laterally and longitudinally. It is more convenient to work in terms of load under one corner of a rectangular loaded area rather than at the center. Dividing the tabular values of C_s by 4 will give the effect for this condition. Stresses from various types of surcharge loadings can be computed using computer solutions (Jumikis 1969, 1971).

3. *Highway Loads*

Pavements designed for heavy truck traffic substantially reduce the pressure transmitted through a wheel to the subgrade, and consequently to underlying sewer pipe. The pressure reduction is so great that generally the live load can be neglected. For heavy duty asphalt or flexible pavements, the reduction will be comparable to that for concrete pavements. The cost savings as a result of the resulting reduction in required pipe strength can be substantial.

For intermediate thicknesses of asphalt or flexible pavements, there is no generally accepted theory for estimating load distribution effects.

Relatively thin pavements do not reduce the pressure transmitted from the wheel to the subgrade to any significant degree. Such pavements are generally considered as unsurfaced roadways for determination of the effect of live loads on buried sewer pipe. A practical method, accepted by AASHTO, is developed in the following paragraphs.

Load Assumptions The maximum highway wheel loads generally considered for design purposes are those specified by AASHTO for HS20 truck and alternate load configurations (Figure 14.20). The critical axle loads for these configurations are carried on dual wheels. The contact area of the dual wheels is assumed to be oval in shape and is approximately equal to the wheel load divided by the tire pressure. Several assumptions are normally made to simplify the evaluation process. For an HS20 wheel load, the contact pressure is assumed equal to the tire pressure. The contact area can be approximated by a rectangle, as shown in Figure 14.21. The possible combinations of load applications

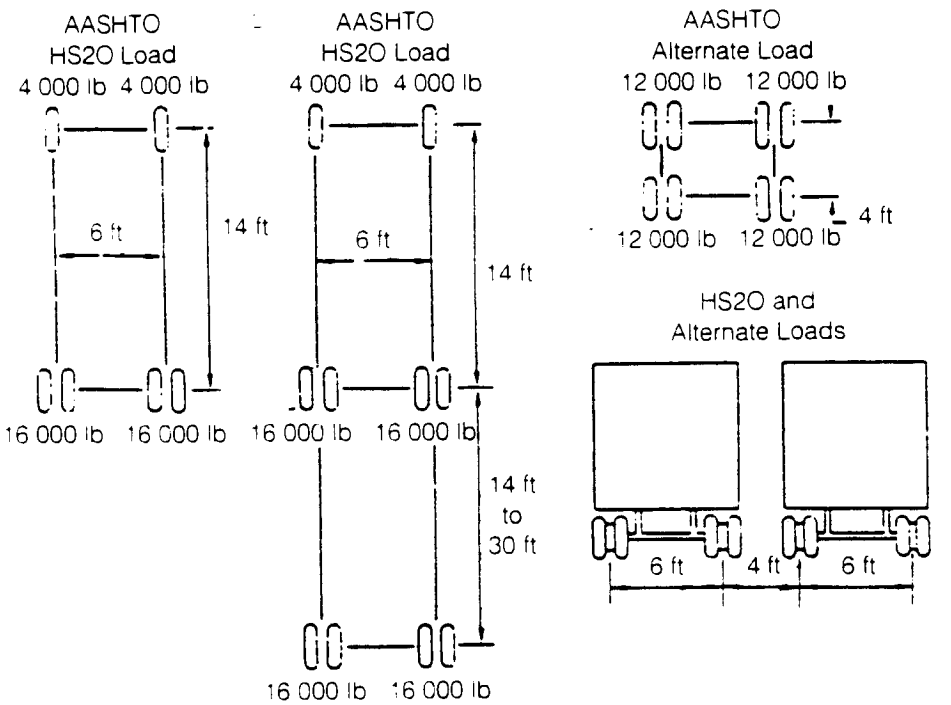


Figure 14.20—Live load spacing (ft × 0.304 8 = m; lb × 0.453 6 = kg) (Am. Concrete Pipe Assoc., 1988).

are a single dual wheel load, two HS20 trucks passing, and the alternate load configuration in the passing mode. For highway loads, AASHTO recommends that the live load be increased by an impact factor when the pipe is less than three feet beneath the pavement surface. The recommended impact factors are presented in Table 14.6.

Load Distribution Assumptions At depths H below the surface, the length and the width of the rectangle are assumed to increase by the value of $1.75H$. The conditions for a single dual wheel, two HS20

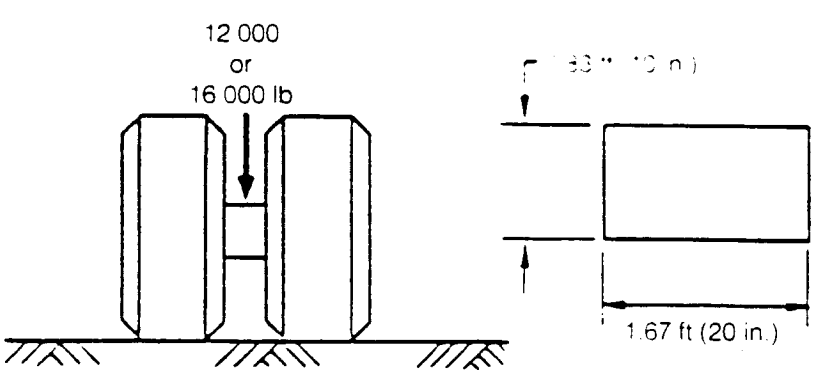


Figure 14.21—Wheel load surface contact area (ft × 0.304 8 = m; in. × 25.4 = mm; lb × 0.453 6 = kg) (Am. Concrete Pipe Assoc., 1988).

TABLE 14.6. Impact Factors for Traffic Loads (AASHTO 1990).

Height of Cover	Impact Factor
0′-0″ to 1′-0″	30%
1′-1″ to 2′-0″	20%
2′-1″ to 2′-11″	10%
3′-0″ and Greater	0

Note: Impact factors recommended by the American Association of State Highway and Transportation Officials in "Standard Specifications for Highway Bridges," Twelfth Edition.

trucks passing, and alternate loads in the passing mode are shown in Figures 14.22, 14.23, and 14.24. The maximum value for average pressure intensity is obtained under various conditions of live load and depth of pipe. As shown in Figures 14.23 and 14.24, at greater depths the distributed loaded areas overlap, and the maximum pressure will develop under either the truck passing or the alternate loads in passing mode conditions. The depths for these transitions is shown in Table 14.7, along with the total loading, and rectangular areas that will produce the critical load effect.

Live Load on Pipe To determine the live load pressure on a buried sewer pipe, the average pressure intensity at the elevation of the outside top of the pipe is calculated. Then the total live load acting on the pipe is calculated based on the average pressure intensity. Finally, the live

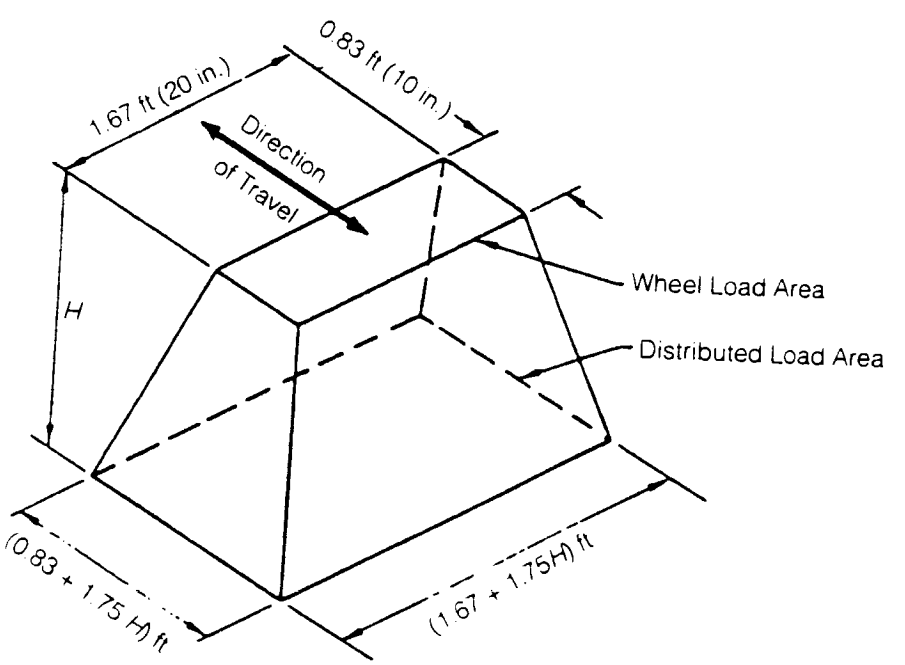


Figure 14.22—Distributed load area—single dual wheel (ft × 0.304 8 = m; in. × 25.4 = mm) (Am. Concrete Pipe Assoc., 1988).

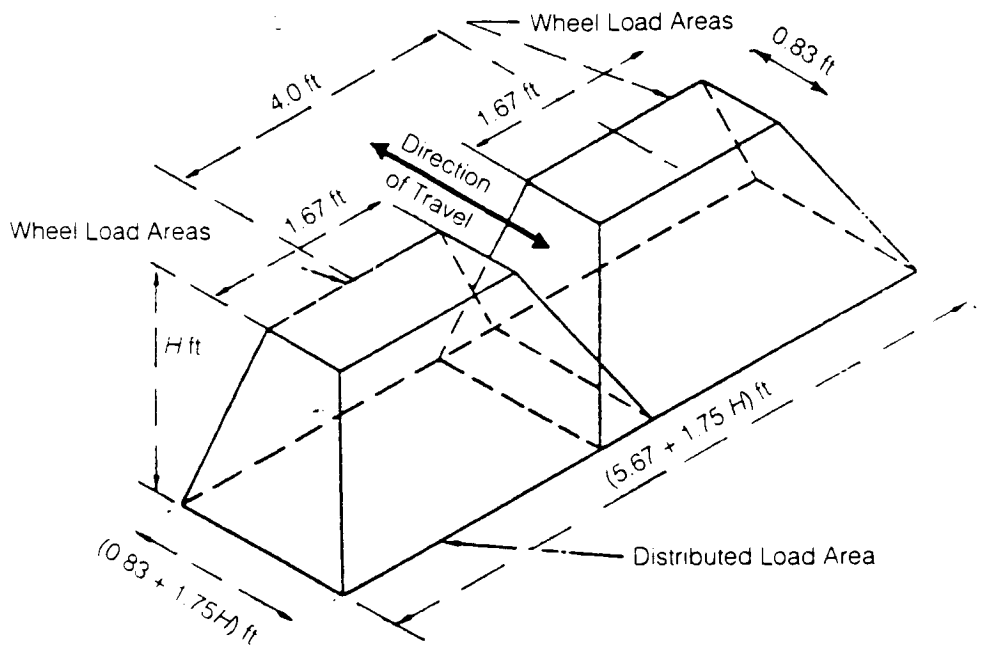


Figure 14.23—Distributed load area—two HS2O trucks passing (ft × 0.304 8 = m) (Am. Concrete Pipe Assoc., 1988).

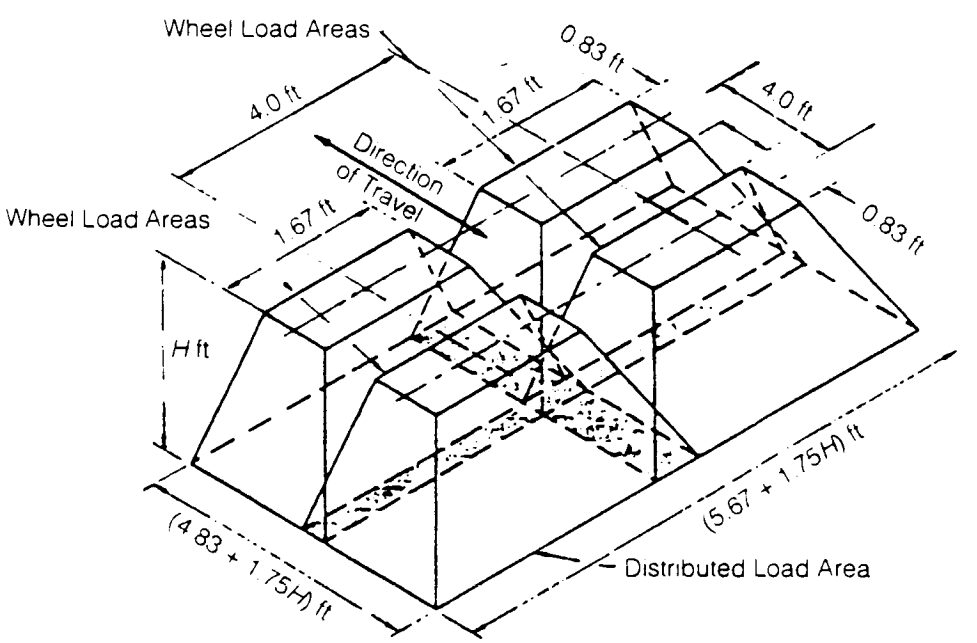


Figure 14.24—Distributed load area—alternate loads in passing mode (ft × 0.304 8 = m) (Am. Concrete Pipe Assoc., 1988).

TABLE 14.7. Critical Loading Configurations (ACPA 1988).

H, Feet	P, Pounds	A_{LL}, Square Feet
H < 1.33	16,000	(0.83 + 1.75H) (1.67 + 1.75H) (See Fig. 4.26)
1.33 ≤ H < 4.10	32,000	(0.83 + 1.75H) (5.67 + 1.75H) (See Fig. 4.27)
4.10 ≤ H	48,000	(4.83 + 1.75H) (5.67 + 1.75H) (See Fig. 4.28)

load pressure per linear foot of pipe is obtained by dividing the total live load on the pipe by the effective supporting length, as shown in Figure 14.25.

The average pressure intensity at the level of the outside top of the pipe is obtained by:

$$w_L = \frac{P(1 + I_f)}{A_{LL}} \tag{14-23}$$

in which w_L is the average pressure intensity, in pounds per square foot; P is the total applied surface wheel loads; A_{LL} is the distributed live load area on the subsoil plane at the outside top of the pipe; and I_f is the impact factor.

Given the average pressure intensity that is critical, the total live load, W_T, is obtained by the following equation:

$$W_T = w_L L S_L \tag{14-24}$$

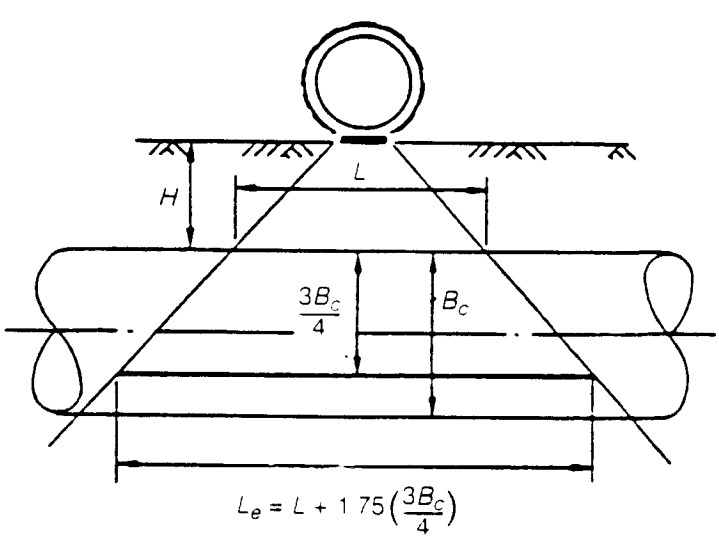


Figure 14.25—Effective supporting length of pipe (Am. Concrete Pipe Assoc., 1988).

in which W_T is the total live load; L is the length of A_{LL} parallel to longitudinal axis of pipe; and S_L is the outside horizontal span of pipe or width of A_{LL} transverse to longitudinal axis of pipe, whichever is less.

A designer is primarily concerned with maximum loads. The most critical pipe location is centered under the distributed load area. The most critical loading can occur either when the longitudinal pipe axis is parallel or is transverse to the direction of truck travel, depending upon the diameter of pipe and depth of cover. Values for L and S_L to be used in Equation 14-24 are obtained from the orientation of the longitudinal pipe axis with respect to the distributed load area. The maximum value of W_T calculated for the two possible orientations, is used in Equation (14-25) to determine the live load, W_L:

$$W_L = \frac{W_T}{L_e} \tag{14-25}$$

in which W_L is the live load on pipe, and L_e is the effective supporting length of pipe.

The buried concrete pipe is assumed to be a beam on a continuous support, and L_e is at a level $B_c/4$ above the bottom of the outside of the pipe, as shown in Figure 14.25. Highway live load effects at depths greater than 10′ below the pavement surface are insignificant, and, as a result, for pipes with cover exceeding that amount, live load effects can be neglected. The following equation is used to obtain L_e:

$$L_e = L + 1.75\left(\frac{3B_c}{4}\right) \tag{14-26}$$

4. *Sewer Pipe Under Airport Pavements*

An important factor in airport operations is the proper functioning of its drainage facilities. Because of relatively shallow covers associated with subsurface drainage for airfields, and the high wheel loads, the effect of aircraft loads on the structural design of underground conduits is more critical than most highway facilities.

The pressure distribution of aircraft wheel loads on any horizontal plane in the soil mass depends on the magnitude and characteristics of the aircraft loads, including tire pressures, the landing gear configuration, the pavement structure, and the subsoil conditions. Larger aircraft have multiple wheel assemblies and dual tandem assemblies to reduce the load concentrations. These cause a combination of loadings that impose overlapping pressures similar to, but greater than, highway loadings.

Major airports are paved with relatively strong surfaces of concrete or asphalt materials, which effectively reduce the load intensity on the

subgrade. The method described in Section 6 can be used for estimating pressure intensities under concrete pavements. For asphalt surfaces, either local engineering practice or a nominal increase in pressure intensity over that estimated for concrete can be used.

There are a large number of small airports with no surfacing beyond a nominal amount to control dust and minimize the destructive effect of windblown aggregate particles. For these, the Boussinesq Solution, which assumes no pavement effect on load distribution, can be used to determine the subsoil pressure intensities from live loads.

5. *Sewer Pipe Under Railway Tracks*

To evaluate the live load effect for railroad loadings, the American Railway Engineering Association recommends the use of a combination of axle loads and axle spacing represented by the Cooper E 80 loading shown in Figure 14.26 and an impact factor ranging from 40% at zero cover to zero percent with 10′ of cover. The series of axle loads and spacings is converted into a uniform load at the bottom of the railroad ties. The pressure intensity on a pipe at various depths and various offsets is then computed based upon the Boussinesq theory. The live load transmitted to a pipe underground is computed by:

$$W_L = Cp_oB_c(1 + I_f) \tag{14-27}$$

in which W_L is the live load transmitted to the pipe; C is the pressure coefficient; p_o is the intensity of the distributed load at the bottom of the ties; B_c is the outside horizontal span of the pipe; and I_f is the impact factor.

This equation is similar to the expressions for highway and airport live loadings in that the pressure intensity on the pipe is equal to the pressure on the surface multiplied by an appropriate coefficient.

A locomotive load is assumed uniformly distributed over an area 8′ by 20′, and the dead load weight of the track structure is assumed to be 200 lbs per linear foot. Live and dead load curves, including impact factors, are plotted in Figure 14.27 as determined from Equation (14-27) for a Cooper E 80 loading. For any given height of cover from the

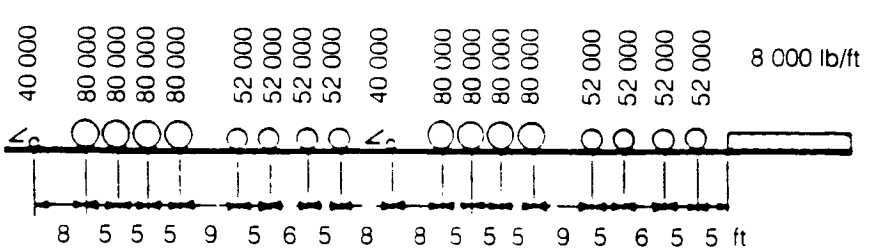


Figure 14.26—Spacing of wheel loads per axle for a Cooper E 80 design loading (lb/ft × 1.488 = kg/m) (Am. Railway Eng. Assoc., 1988).

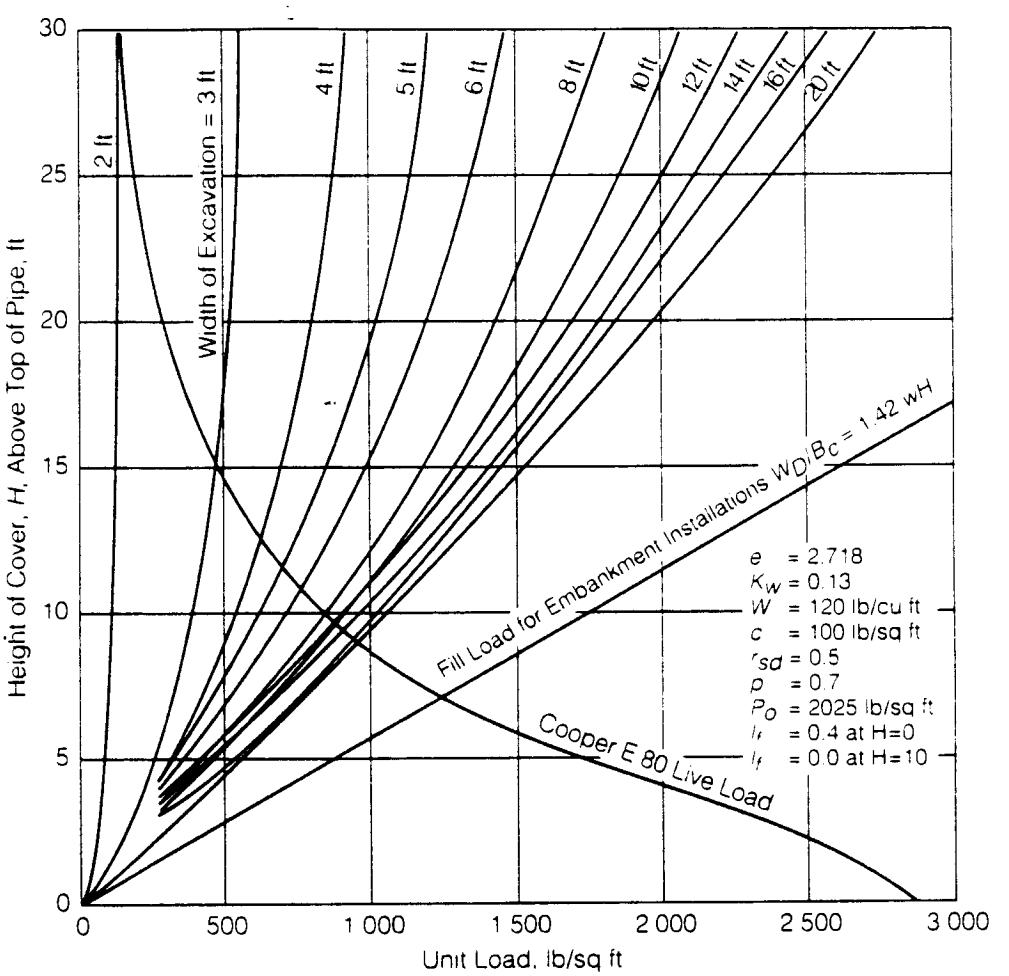


Figure 14.27—Live and dead loads on pipe installed under railroads (ft × 0.304 8 = m; lb/cu ft × 16.02 = kg/m³; lb/sq ft × 47.88 = kg/m²) (Am. Concrete Pipe Assoc., 1988).

top of the pipe to the bottom of the ties, the live load can be read directly from Figure 14.27. To obtain the live load per linear foot, it is necessary to multiply the unit load from Figure 14.27 by the outside span of the pipe, B_c, in feet.

6. *Sewer Pipe Under Rigid Pavement*

Based upon the early work of Westergaard and others, the Portland Cement Association (PCA) developed a method to determine the vertical pressure on buried pipe due to wheel loads applied to concrete pavements (PCA 1951). The PCA approach assumes that both the slab and the earth are elastic materials, the compression of the soil is proportional to the deflection of the slab, and the effect of the load radiates equally in all directions. The following equation for the pressure intensity on concrete pipe under concrete pavements was developed:

R0021440

$$p_{(H,X)} = \frac{CP}{R_s^2} \tag{14-28}$$

in which C is the pressure coefficient, dependent on H, X, and R_s; P is the wheel load; R_s is the radius of stiffness of the rigid pavement; and $p_{(H,X)}$ is the vertical pressure intensity at any horizontal distance, X, and any vertical distance, H, within the soil mass.
R_s is further defined as:

$$R_s = \sqrt[4]{\frac{Eh^3}{12(1 - \mu^2)k}} \tag{14-29}$$

in which E is the modulus of elasticity of concrete, in pounds per square inch; h is the thickness of concrete pavement; μ is the Poisson's ratio of concrete (assumed a constant 0.15); and κ is the modulus of subgrade reaction, pounds per cubic inch.

Equation (14-28) has been solved for various conditions to simplify the task of estimating the live load. The results are included in Tables 14.8 through 14.12 in the form of pressure coefficients for various values of H, X, and R_s. Equation (14-29) also has been solved for a range of conditions and R_s is tabulated for various values of h and κ in Table 14.13. Figure 14.28 provides a basis for estimating the modulus of subgrade reaction using as a basis the various soil classification systems.

The values given in the tables are used to determine the pressures on horizontal planes at depth H in a semi-infinite elastic body from a single wheel and two wheels at various spacings on the pavement surface. The coefficient values in Tables 14.9 through 14.11 are larger, for any given fill height, than the coefficient values presented for a single wheel (Table 14.8). Thus, the combined pressure on any horizontal plane for wheels at a spacing of $2.4R_s$ or less is greater than that for a single wheel. For the greater wheel spacing of $3.2R_s$, the coefficient values in Table 14.12 are greater than for a single load (Table 14.8), except close to the pavement. Therefore, for a wheel spacing of $3.2R_s$, the combined pressure should be used except where the pipe is very close to the pavement subgrade elevation, when the single wheel load pressure can be used. For wheel spacing greater than $3.2R_s$, the combined pressure of two wheels will never be more than the pressure from a single wheel.

The presence of a pipe introduces a boundary condition which, theoretically, creates a case simulating that of an elastic layer of depth H resting on a rigid base. Although pressures based on this concept would be slightly higher than those included in the tabular coefficients, variability of the elastic properties of the backfill over the pipe, the rigidity of the pipe, and the inadequacies of the theory are such that precise computations are not justified.

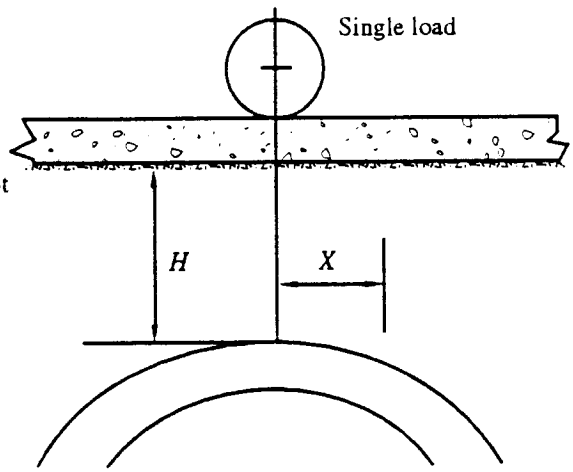
TABLE 14.8. Pressure Coefficients for a Single Load (PCA 1951).

Values of C

$p = \frac{CP}{R_s^2}$ pounds per square foot

P = wheel load, pounds

R_s = radius of stiffness of pavement slab, feet



$\frac{H}{R_s}$	X/R_s										
	0.0	0.4	0.8	1.2	1.6	2.0	2.4	2.8	3.2	3.6	4.0
0.0	.113	.105	.089	.068	.048	.032	.020	.011	.006	.002	.000
0.4	.101	.095	.082	.065	.047	.033	.021	.011	.004	.001	.000
0.8	.089	.084	.074	.061	.045	.033	.022	.012	.005	.002	.001
1.2	.076	.072	.065	.054	.043	.032	.022	.014	.008	.005	.003
1.6	.062	.059	.054	.047	.039	.030	.022	.016	.011	.007	.005
2.0	.051	.049	.046	.042	.035	.028	.022	.016	.011	.008	.006
2.4	.043	.041	.039	.036	.030	.026	.021	.016	.011	.008	.006
2.8	.037	.036	.033	.031	.027	.023	.019	.015	.011	.009	.006
3.2	.032	.030	.029	.026	.024	.021	.018	.014	.011	.009	.007
3.6	.027	.026	.025	.023	.021	.019	.016	.014	.011	.009	.007
4.0	.024	.023	.022	.020	.019	.018	.015	.013	.011	.009	.007
4.4	.020	.020	.019	.018	.017	.015	.014	.012	.010	.009	.007
4.8	.018	.017	.017	.016	.015	.013	.012	.011	.009	.008	.007
5.2	.015	.015	.014	.014	.013	.012	.011	.010	.008	.007	.006
5.6	.014	.013	.013	.012	.011	.010	.010	.009	.008	.007	.006
6.0	.012	.012	.011	.011	.010	.009	.009	.008	.007	.007	.006
6.4	.011	.010	.010	.010	.009	.008	.008	.007	.007	.006	.005
6.8	.010	.009	.009	.009	.008	.008	.007	.007	.006	.006	.005
7.2	.009	.008	.008	.008	.008	.007	.007	.006	.006	.006	.005
7.6	.008	.008	.008	.007	.007	.007	.006	.006	.006	.005	.005
8.0	.007	.007	.007	.007	.006	.006	.006	.006	.005	.005	.005

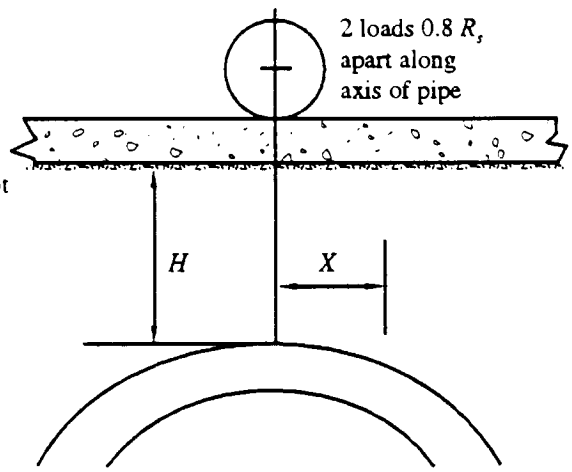
TABLE 14.9. Pressure Coefficients for Two Loads Spaced $0.8R_s$ Apart (PCA 1951).

Values of C

$p = \frac{CP}{R_s^2}$ pounds per square foot

P = wheel load, pounds

R_s = radius of stiffness of pavement slab, feet



$\frac{H}{R_s}$	X/R_s										
	0.0	0.4	0.8	1.2	1.6	2.0	2.4	2.8	3.2	3.6	4.0
0.0	.210	.198	.168	.130	.092	.062	.038	.022	.011	.004	.000
0.4	.190	.181	.156	.126	.092	.064	.040	.023	.010	.002	.000
0.8	.168	.160	.140	.117	.088	.063	.042	.024	.010	.003	.001
1.2	.144	.139	.124	.106	.083	.062	.043	.027	.013	.007	.004
1.6	.118	.115	.105	.094	.076	.060	.044	.030	.020	.014	.009
2.0	.098	.095	.089	.081	.070	.056	.043	.032	.023	.017	.012
2.4	.083	.080	.076	.069	.061	.050	.040	.031	.023	.017	.012
2.8	.071	.069	.066	.060	.053	.045	.037	.029	.022	.017	.012
3.2	.061	.059	.057	.052	.046	.040	.034	.028	.022	.017	.013
3.6	.052	.051	.049	.046	.041	.036	.032	.027	.022	.018	.014
4.0	.045	.044	.042	.040	.037	.034	.030	.026	.022	.018	.015
4.4	.039	.038	.037	.035	.033	.030	.027	.024	.021	.017	.015
4.8	.034	.034	.033	.031	.029	.027	.024	.021	.019	.016	.014
5.2	.030	.029	.028	.027	.025	.023	.021	.019	.017	.015	.013
5.6	.026	.026	.025	.024	.022	.021	.019	.018	.016	.014	.012
6.0	.023	.023	.022	.021	.020	.019	.017	.016	.015	.013	.011
6.4	.021	.021	.020	.019	.018	.017	.016	.015	.014	.012	.011
6.8	.019	.019	.018	.018	.017	.016	.015	.014	.013	.012	.010
7.2	.017	.017	.016	.016	.015	.014	.013	.013	.012	.011	.010
7.6	.016	.015	.015	.015	.014	.013	.012	.012	.011	.010	.009
8.0	.014	.014	.014	.013	.013	.012	.012	.011	.010	.010	.009

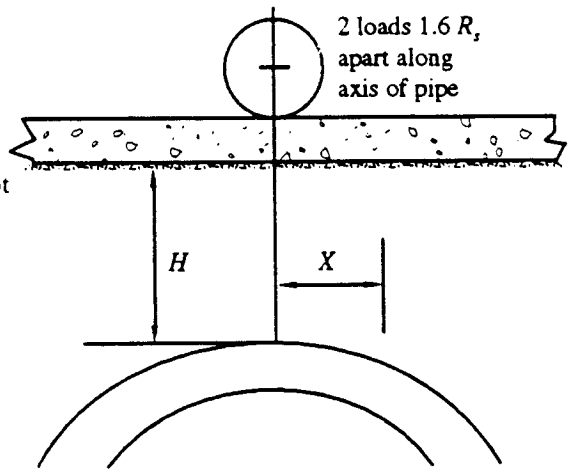
TABLE 14.10. Pressure Coefficients for Two Loads Spaced $1.6R_s$ Apart (PCA 1951).

Values of C

$p = \frac{CP}{R_s^2}$ pounds per square foot

P = wheel load, pounds

R_s = radius of stiffness of pavement slab, feet



$\frac{H}{R_s}$	X/R_s										
	0.0	0.4	0.8	1.2	1.6	2.0	2.4	2.8	3.2	3.6	4.0
0.0	.178	.167	.142	.112	.080	.054	.034	.019	.009	.004	.000
0.4	.164	.156	.136	.109	.080	.056	.036	.019	.008	.002	.000
0.8	.147	.141	.126	.103	.078	.057	.037	.020	.008	.002	.001
1.2	.128	.124	.106	.094	.074	.056	.039	.023	.012	.006	.004
1.6	.108	.105	.097	.082	.070	.054	.040	.028	.019	.014	.009
2.0	.092	.090	.084	.075	.065	.052	.040	.030	.022	.017	.012
2.4	.079	.076	.072	.065	.056	.047	.038	.029	.022	.017	.012
2.8	.068	.066	.062	.058	.050	.043	.035	.028	.022	.017	.012
3.2	.058	.056	.054	.050	.044	.038	.032	.027	.022	.017	.012
3.6	.050	.049	.047	.044	.040	.035	.030	.026	.022	.017	.013
4.0	.043	.042	.041	.039	.036	.033	.030	.026	.022	.018	.015
4.4	.038	.037	.036	.034	.032	.029	.026	.023	.020	.016	.014
4.8	.033	.032	.031	.030	.028	.026	.024	.021	.018	.015	.013
5.2	.029	.028	.027	.026	.025	.023	.021	.019	.016	.014	.012
5.6	.025	.025	.024	.023	.022	.020	.019	.017	.015	.013	.012
6.0	.023	.022	.022	.021	.019	.018	.017	.016	.014	.013	.011
6.4	.020	.020	.019	.019	.018	.016	.015	.015	.013	.012	.011
6.8	.018	.018	.018	.017	.016	.015	.014	.013	.012	.011	.010
7.2	.017	.016	.016	.015	.015	.014	.013	.013	.012	.011	.010
7.6	.015	.015	.014	.014	.014	.013	.012	.012	.011	.010	.010
8.0	.014	.014	.013	.013	.013	.012	.011	.011	.010	.010	.009

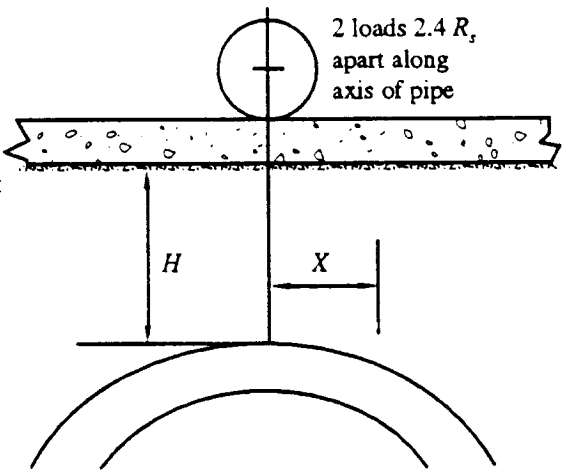
TABLE 14.11. Pressure Coefficients for Two Loads Spaced 2.4 R_s Apart (PCA 1951).

Values of C

$p = \frac{CP}{R_s^2}$ pounds per square foot

P = wheel load, pounds

R_s = radius of stiffness of pavement slab, feet



$\frac{H}{R_s}$	X/R_s										
	0.0	0.4	0.8	1.2	1.6	2.0	2.4	2.8	3.2	3.6	4.0
0.0	.137	.130	.112	.088	.065	.044	.028	.014	.007	.003	.000
0.4	.130	.125	.109	.087	.066	.047	.028	.013	.005	.001	.000
0.8	.121	.117	.104	.085	.066	.048	.030	.014	.006	.002	.001
1.2	.109	.105	.096	.079	.064	.048	.033	.018	.012	.006	.005
1.6	.095	.092	.084	.072	.060	.047	.035	.025	.018	.012	.009
2.0	.083	.081	.077	.068	.057	.046	.035	.026	.020	.015	.010
2.4	.070	.069	.065	.059	.052	.044	.034	.026	.020	.015	.011
2.8	.062	.060	.058	.053	.046	.039	.033	.027	.020	.015	.011
3.2	.053	.052	.050	.046	.041	.035	.032	.026	.020	.016	.012
3.6	.046	.045	.044	.042	.038	.034	.030	.026	.021	.017	.013
4.0	.040	.040	.039	.037	.035	.032	.029	.025	.021	.017	.014
4.4	.036	.035	.034	.033	.031	.028	.025	.022	.019	.016	.013
4.8	.031	.031	.030	.029	.027	.025	.022	.020	.017	.015	.012
5.2	.027	.027	.026	.025	.024	.022	.020	.018	.016	.014	.012
5.6	.024	.023	.023	.022	.021	.020	.018	.017	.015	.013	.011
6.0	.022	.021	.021	.020	.019	.018	.017	.015	.014	.012	.011
6.4	.019	.019	.019	.018	.017	.016	.015	.014	.013	.012	.010
6.8	.018	.017	.017	.016	.016	.015	.014	.013	.012	.011	.010
7.2	.016	.016	.016	.015	.014	.014	.013	.012	.011	.010	.009
7.6	.015	.014	.014	.014	.013	.013	.012	.011	.011	.010	.009
8.0	.013	.013	.013	.013	.012	.012	.011	.011	.010	.009	.009

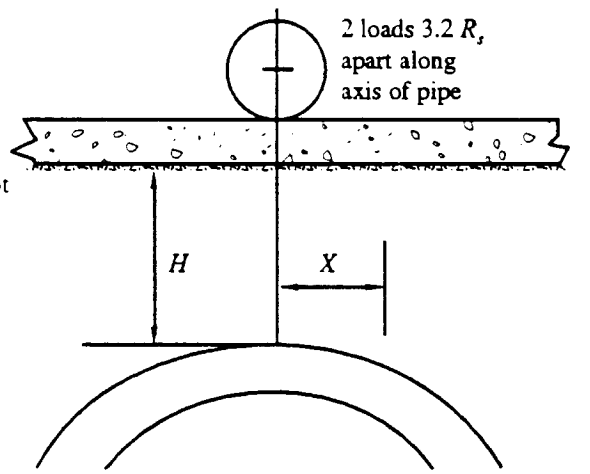
TABLE 14.12. Pressure Coefficients for Two Loads Spaced 3.2 R_S Apart (PCA 1951).

Values of C

$p = \frac{CP}{R_s^2}$ pounds per square foot

P = wheel load, pounds

R_s = radius of stiffness of pavement slab, feet



$\frac{H}{R_s}$	X/R_s										
	0.0	0.4	0.8	1.2	1.6	2.0	2.4	2.8	3.2	3.6	4.0
0.0	.097	.093	.080	.065	.048	.032	.020	.011	.004	.000	.000
0.4	.096	.092	.079	.067	.050	.034	.020	.010	.003	.000	.000
0.8	.092	.088	.078	.066	.051	.036	.021	.010	.003	.000	.000
1.2	.086	.082	.074	.066	.050	.038	.025	.014	.007	.003	.001
1.6	.077	.075	.068	.060	.049	.039	.030	.021	.015	.011	.007
2.0	.070	.068	.063	.057	.048	.040	.031	.023	.017	.013	.009
2.4	.061	.060	.056	.051	.045	.038	.030	.023	.017	.013	.010
2.8	.056	.054	.052	.048	.042	.036	.029	.023	.018	.013	.010
3.2	.048	.046	.044	.041	.037	.032	.028	.023	.018	.014	.010
3.6	.043	.041	.040	.038	.034	.030	.027	.022	.019	.015	.012
4.0	.038	.037	.036	.035	.032	.029	.026	.022	.019	.016	.013
4.4	.033	.033	.032	.031	.029	.027	.024	.020	.018	.015	.013
4.8	.029	.029	.028	.027	.025	.023	.021	.018	.016	.014	.012
5.2	.025	.025	.025	.024	.022	.021	.019	.017	.015	.013	.012
5.6	.022	.022	.022	.021	.020	.018	.017	.016	.014	.012	.011
6.0	.020	.020	.020	.020	.020	.017	.016	.015	.013	.011	.011
6.4	.018	.018	.018	.018	.018	.016	.015	.014	.012	.011	.010
6.8	.016	.016	.016	.016	.016	.014	.014	.013	.012	.010	.010
7.2	.015	.015	.015	.015	.015	.013	.013	.012	.011	.010	.009
7.6	.014	.014	.013	.013	.013	.012	.012	.011	.010	.009	.009
8.0	.013	.013	.012	.012	.012	.011	.011	.010	.010	.009	.008

TABLE 14.13. Values of Radius of Stiffness, R_s (PCA 1951).

h (in.)	Values of k								
	50	100	150	200	250	300	350	400	500
6	34.84	29.30	26.47	24.63	23.30	22.26	21.42	20.72	19.59
6.5	36.99	31.11	28.11	26.16	24.74	23.64	22.74	22.00	20.80
7	39.11	32.89	29.72	27.65	26.15	24.99	24.04	23.25	21.99
7.5	41.19	34.63	31.29	29.12	27.54	26.32	25.32	24.49	23.16
8	43.23	36.35	32.85	30.57	28.91	27.62	26.58	25.70	24.31
8.5	45.24	38.04	34.37	31.99	30.25	28.91	27.81	26.90	25.44
9	47.22	39.71	35.88	33.39	31.58	30.17	29.03	28.08	26.55
9.5	49.17	41.35	37.36	34.77	32.89	31.42	30.23	29.24	27.65
10	51.10	42.97	38.83	36.14	34.17	32.65	31.42	30.39	28.74
10.5	53.01	44.57	40.28	37.48	35.45	33.87	32.59	31.52	29.81
11	54.89	46.16	41.71	38.81	36.71	35.07	33.75	32.64	30.87
11.5	56.75	47.72	43.12	40.13	37.95	36.26	34.89	33.74	31.91
12	58.59	49.27	44.52	41.43	39.18	37.44	36.02	34.84	32.95
12.5	60.41	50.80	45.90	42.72	40.40	38.60	37.14	35.92	33.97
13	62.22	52.32	47.27	43.99	41.61	39.75	38.25	36.99	34.99
13.5	64.00	53.82	48.63	45.26	42.80	40.89	39.35	38.06	35.99
14	65.77	55.31	49.98	46.51	43.98	42.02	40.44	39.11	36.99
14.5	67.53	56.78	51.31	47.75	45.16	43.15	41.51	40.15	37.97
15	69.27	58.25	52.63	48.98	46.32	44.26	42.58	41.19	38.95
15.5	70.99	59.70	53.94	50.20	47.47	45.36	43.64	42.21	39.92
16	72.70	61.13	55.24	51.41	48.62	46.45	44.70	43.23	40.88
16.5	74.40	62.56	56.53	52.61	49.75	47.54	45.74	44.24	41.84
17	76.08	63.98	57.81	53.80	50.88	48.61	46.77	45.24	42.78
17.5	77.75	65.38	59.08	54.98	52.00	49.68	47.80	46.23	43.72
18	79.41	66.78	60.35	56.16	53.11	50.74	48.82	47.22	44.66
19	82.70	69.54	62.84	58.48	55.31	52.84	50.84	49.17	46.51
20	85.95	72.27	65.30	60.77	57.47	54.92	52.84	51.10	48.33
21	89.15	74.97	67.74	63.04	59.62	56.96	54.81	53.01	50.13
22	92.31	77.63	70.14	65.28	61.73	58.98	56.75	54.89	51.91
23	95.44	80.27	75.52	67.49	63.83	60.98	58.68	56.75	53.67
24	98.54	82.86	74.87	69.68	65.90	62.96	60.58	58.59	55.41

$$R_s = \sqrt[4]{\frac{Eh^3}{12\,(1 - \mu^2)\,k}}$$

where: $E = 4{,}000{,}000$ psi, $\mu = 0.15$ $\quad R_s = 24.1652 \sqrt[4]{\frac{h^3}{k}}$

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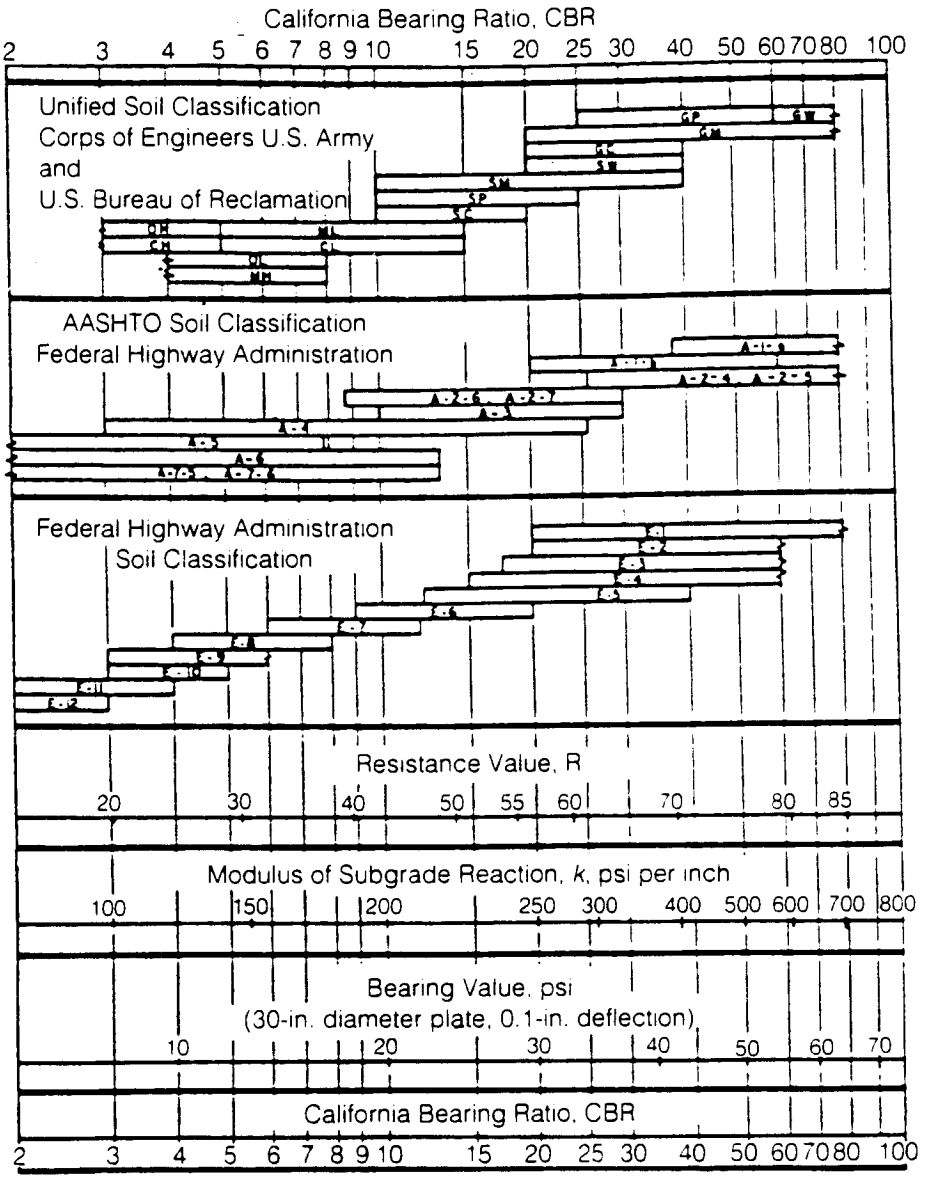

Figure 14.28—Approximate interrelationships of soil classifications and bearing values (in. × 25.4 = mm; psi × 6895 = Pa) (Am. Concrete Pipe Assoc., 1988).

D. Pipe Bedding and Backfilling

1. General Concepts

The ability of a sewer pipe to safely support the calculated soil load depends not only on its inherent strength but also on the distribution of the bedding reaction and on the lateral pressure acting against the sides of the sewer pipe.

Construction of the sewer pipe/soil system focuses attention on the pipe zone, which is made up of five specific areas; foundation, bedding,

haunching, initial backfill, and final backfill (see Figure 14.29 for definitions and limits of these five areas). Note that all of these areas are not necessarily referred to in all pipe design standards. The discussion in this section is in general terms and is intended to describe the effect of the various areas on the pipe-soil system.

For all sewer pipe materials, the calculated vertical load is assumed to be uniformly distributed over the width of the pipe. This assumption originated in Marston's work; the assumption is part of the bedding factors developed by Spangler and presented in this chapter. Many years of field experience indicate that the assumption results in conservative designs.

The load-carrying capacity of sewer pipes of all materials is influenced by the sewer pipe/soil system, although the importance of the specific areas may vary with different pipe materials. Detailed information on pipe bedding classes is contained in the various ANSI and ASTM Specifications or industry literature for each material. The design engineer should consult the applicable specification or literature for information to be used in design.

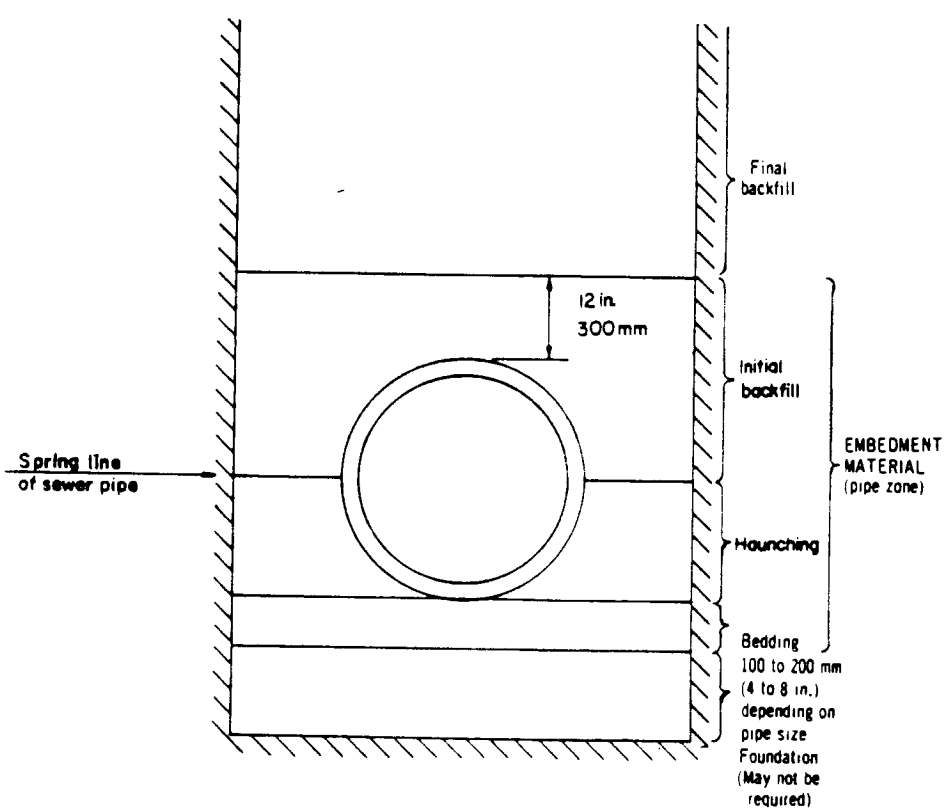

Figure 14.29—Trench cross section illustrating terminology (ASCE, 1982)

2. *Foundation*

The foundation provides the base for the sewer pipe-soil system. In trench conditions, the total weight of the pipe and soil backfill will normally be no more than the weight of the excavated soil. In this case, foundation pressures are not increased from the initial condition, and the designer should be concerned primarily with the presence of unsuitable soils, such as peat or other highly organic or compressible soils, and with maintaining a stable trench bottom. If the full benefit of the bedding is to be gained, the bottom of the trench or embankment must be stable.

3. *Bedding*

Bedding provides the interface between the pipe and its foundation, and has an important influence on the distribution of the reaction against the bottom of the sewer pipe. Bedding therefore influences the supporting strength of the pipe as installed. Some research (Griffith and Keeney 1967; Sikora 1980) has indicated that a well-graded stone is a suitable material for sewer pipe bedding, and that the most important property of any bedding material is the degree to which the pipe can settle into the bedding. A crushed material that is self-compacting and will interlock should be avoided.

Even though larger particle sizes give greater stability, the maximum size and shape of granular embedment should also be related to the pipe material and the recommendations of the manufacturer. For example, sharp angular embedment material larger than 0.5–0.75 inch should not be used against corrosion protection coatings. For small sewer pipes, the maximum size should be limited to about 10% of the pipe diameter.

Soil classifications under the Unified Soil Classification System, including manufactured materials, are grouped into five broad categories according to their ability to develop an interacting sewer pipe-soil system (Table 14.14). These soil classes are described in ASTM D 2321.

In general, crushed stone or gravel meeting the requirements of ASTM Designation C33, Gradation 67 [0.75 in. to No. 4)] will provide the most satisfactory sewer pipe bedding.

In some locations, the natural soils at the level of the bottom of the sewer pipe may be sands of suitable grain size and density to serve as both foundation and bedding for the pipe. In such situations it may not be necessary to remove and replace these soils with the special bedding materials described above.

4. *Haunching*

The soil placed at the sides of a pipe from the bedding up to the spring line is the haunching. The care with which this material is placed has a significant influence on the performance of the sewer pipe, particularly in the space just above the bedding. Poorly compacted material

TABLE 14.14. Soil Classifications* (ASCE 1982).

Soil Class (1)	Group Symbol (2)	Typical Names (3)	Comments (4)
I		Crushed rock	angular, 6–40 mm
II	GW GP SW SP	Well graded gravels Poorly graded gravels Well graded sands Poorly graded sands	40 mm maximum
III	GM GC SM SC	Silty gravels Clayey gravels Silty sands Clayey sands	
IV	MH, ML CH, CL	Inorganic silts Inorganic clays	Not recommended for bedding, haunching or initial backfill
V	OL, OH PT	Organic silts and clays Peat	

*For a more detailed description, see ASTM D2321—"Recommended Practice for Underground Installation of Flexible Thermoplastic Sewer Pipe."
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Note: 1mm = 0.039 in.

in this space will result in a concentration of reaction at the bottom of the pipe.

For flexible pipe, compaction of the haunching material is essential. For rigid pipe, compaction can ensure better distribution of the forces on the pipe. Material used for sewer pipe haunching should be shovel sliced or otherwise placed to provide uniform support for the pipe barrel and to fill completely all voids under the pipe. Because of space limitations, haunching material is often compacted manually. Results should be checked to verify that the bedding or installation criteria are achieved.

Material used for haunching may be crushed stone or sand, or a well-graded granular material of intermediate size. If crushed stone is used, it should be subject to the same size limitations and cautions regarding use against corrosion protection coatings. Sand should not be used if the pipe zone area is subject to a fluctuating groundwater table or where there is a possibility of the sand migrating into the pipe bedding or trench walls, unless a geotextile is incorporated in the installation.

5. *Initial Backfill*

Initial backfill is the material that covers the sewer pipe and extends from the haunching to some specific point (generally 6–12") above the

top of the pipe, depending on the class of bedding and type of pipe material. Its function is to protect the pipe from damage by subsequent backfill, and to ensure the uniform distribution of load over the top of the pipe. For rigid pipe, the structural performance is primarily based on the haunching material; whereas for flexible pipe, compaction of the initial backfill is critical to performance.

The initial backfill is usually not mechanically tamped or compacted, since such work may damage the sewer pipe, particularly if done over the crown of the pipe. Therefore it should be a material that will develop a uniform and, for flexible sewer pipe, relatively high density with little compactive effort. Initial backfill should consist of suitable granular material, but for rigid pipe not necessarily as select a material as that used for bedding and haunching. Clay materials requiring mechanical compaction should not be used for initial backfill.

The fact that little compaction effort is used on the initial backfill should not lead to carelessness in choice or placement of material. Particularly for large sewer pipes, care should be taken in placing both the initial backfill and final backfill over the crown to avoid damage to the sewer pipe.

6. Final Backfill

The choice of material and placement methods for final backfill are related to the construction site and (generally) not related to the design of the sewer pipe. Under special embankment conditions or induced trench conditions, final backfill may play an important part in the sewer pipe design. However, for most trench installations, final backfill does not affect the pipe design.

The final backfill of trenches in traffic areas, such as under improved existing surfaces, is usually composed of material that is easily densified to minimize future settlement. In underdeveloped areas, final backfill often will consist of the excavated material placed with little compaction and left mounded over the trench to allow for future settlement. Studies have indicated that with some soils, this settlement may continue for over 10 years.

Trench backfilling should be done in such a way as to prevent dropping of material directly on top of a sewer pipe through any great vertical distance. When placing material with a bucket, the bucket should be lowered so that the shock of falling earth will not cause damage.

E. Design Safety Factor and Performance Limits

1. General Concepts

In the design of sewer pipes, the selection of a factor of safety is an essential element of the structural design requirements. When these requirements are defined for a given construction material, the most severe or maximum performance limits of that material as related to

the proposed service or design application limit must be determined, and the ratio of those two limits is the "factor of safety." Maximum performance limits and design values may be defined in terms of initial or long-term strength, stress, strain, or product deformation depending on the characteristics of the material under consideration. Typical design values are presented in AASHTO standards (AASHTO undated). The selected factor of safety is applied to the maximum performance limit to calculate a lower value, which is then used as a design or service performance value.

The selection of a desired factor of safety is essentially based on a risk value assessment which must relate to the specific conditions anticipated in an application, the failure mode of the construction material, and the potential cost of system failure. Factors of safety compensate for unexpected construction deficiencies. They should not be relied on to compensate for poor construction practice or for inadequate inspection. Properly established design performance values, including adequate factors of safety, must be realized in installation and operation to provide reasonable assurance of acceptable long-term system performance.

The relationship between safety factors and design performance values is similar for rigid and flexible sewer pipe. However, an understanding of the differences between design requirements for rigid sewer pipe and the requirements for flexible sewer pipe system design is important.

2. *Rigid Sewer Pipe*

Design performance limits for rigid sewer pipes generally are expressed in terms of strength under load. There are two alternative methods of determining the service strength of reinforced concrete sewer pipe: by "strength and design" analysis, or by testing. The direct design method more accurately represents the relative performance of pipe in the field under the service load. However, testing is generally used for the indirect design types of precast or prefabricated rigid pipe.

Indirect design strengths of rigid sewer pipe usually are measured in terms of the ultimate three-edge bearing strength, and of ultimate and 0.01" crack, three-edge bearing strengths for reinforced concrete sewer pipe. A safety factor of 1.0 should be applied to the 0.01" crack load for reinforced concrete sewer pipe, and a safety factor of 1.25–1.50 should be applied to the specified minimum ultimate three-edge bearing strength to determine the working strength for other rigid pipes. Common practice is to use a factor of safety of 1.25 for the ultimate load of reinforced concrete sewer pipe, and up to 1.50 for vitrified clay tested in three-edge bearing. Such safety factors relate to test loads that concentrate maximum shear and flexure at the same point. They are very conservative and do not represent ultimate supporting strength in the buried condition.

3. *Flexible Sewer Pipe*

Design performance limits for flexible sewer pipe may be expressed in terms of stress or strain in the pipe wall, crushing or buckling in the pipe wall, or deflection. The most common limitation is deflection. The deflection limitation is established as a design performance limit to provide a factor of safety against structural failure or any type of distress that might tend to limit the service life of the pipe. This design limit will vary with different pipe materials and the pipe manufacturing process. Pipes must be able to deflect without cracking, liner failure, joint leaks, excessive strain or other distress, and they should be designed with a reasonable factor of safety.

Certain types of plastic pipes are subject to strain deterioration. There is a limiting strain which, if exceeded over a period of time, will eventually result in cracking of the pipe wall, which is called environmental stress cracking (ESC). AASHTO Standards (AASHTO undated) provide design values for this limiting strain.

4. *Recommendations for Field Procedures*

The factor of safety against ultimate collapse of sewer pipe is about the same as that used in the design of most engineered structures. However, the design of sewer pipe is based on calculated loads, bedding factors, and experimental factors, which are less well-defined than the dead and live loads used in building design. It is therefore important that the loads imposed on the sewer pipe not exceed the design loads.

To obtain this objective, the following procedures are recommended:

(a) Specifications. Construction specifications should set forth limits for the width of trench at the top of sewer pipe. The width limits should take into account the minimum allowable width for each class of sewer pipe and bedding to be used. Where the depth is such that a positive projecting condition will be obtained, maximum width should be specified as unlimited unless the width must be controlled for some reason other than to meet structural requirements of the sewer pipe. Appropriate corrective measures should be specified in the event the maximum allowable width is exceeded. These measures may include provision for a higher class of bedding or stronger pipe. Maximum allowable construction live loads should be specified for various depths of cover if appropriate.

(b) Inspection. Construction should be observed by an experienced engineer, or by an inspector who reports to a competent field engineer.

(c) Testing. Sewer pipe testing should be under the supervision of a reliable testing laboratory, and close liaison should be maintained between the laboratory and the field engineer.

(d) Field Conditions. The field engineer should be furnished with sufficient design data to enable him to evaluate unforeseen conditions intelligently. The field engineer should be instructed to confer with the design engineer if design changes appear advisable.

(e) Sheeting. Where sheeting is to be removed, pulling should be done in stages. The space formerly occupied by the sheeting must be backfilled completely, and field inspection should verify this.

Effect of Trench Sheeting Because of the various alternative methods employed in sheeting trenches, generalizations on the proper construction procedure follow to ensure that the design load is not exceeded are risky and dangerous. Each method of sheeting and bracing should be studied separately. The effect of a particular system on the sewer pipe load, as well as the consequences of removing the sheeting or the bracing, must be estimated.

It is difficult to obtain satisfactory filling and compaction of the void left when wood sheeting is pulled. Wood sheeting driven alongside the sewer pipe should be cut off and left in place to an elevation of 1.5′ above the top of the sewer pipe.

If granular materials are used for backfill it is possible to fill and compact the voids left by the wood sheeting if the material is placed in lifts and jetted as the sheeting is pulled. If cohesive materials are used for backfill, a void will be left after pulling the wood sheets and the full weight of the prism of earth contained between the sheeting will come to bear on the sewer pipe.

Skeleton sheeting or bracing should be cut off and left in place to an elevation of 1.5′ over the top of the sewer pipe if removal of the trench support might cause a collapse of the trench wall and a widening of the trench at the top of the conduit. Entire skeleton sheeting systems should be left in place if removal would cause collapse of the trench before backfill can be placed.

Where steel soldier beams with horizontal lagging between the beam flanges are used for sheeting trenches, efforts to reclaim the steel beams before the trench is backfilled may damage pipe joints. It is recommended that use of this type of sheeting be allowed only on stipulation that the beams be pulled after backfilling and the lagging be left in place.

Steel sheeting may be used and reused many times, and the relative economy of this type of sheeting compared with timber or timber and soldier beams should be explored. Because of the thinness of the sheeting, it is often feasible to achieve reasonable compaction of backfill so that the steel sheeting may be withdrawn with about the same factor of safety against settlement of the surfaces adjacent to the trench as that for other types of sheeting left in place.

Trench Boxes Several backfilling techniques are possible when a trench box is used. Granular material can be placed between the box and the trench wall immediately after placing the box, and the box is advanced by lifting slightly before moving forward. Boxes can be made with a step at the rear, which makes the trench wall accessible for compacting the embedment against the walls. Also, the box can be used where the sewer pipe is laid in a small sub-trench, and the box is used only in

the trench above the top of the sewer pipe. Advancement of the box must be done carefully to avoid pulling pipe joints apart.

Pipe Bedding and Embedment To assure that the sewer pipe is properly bedded or embedded it is suggested that compaction tests be made at selected or critical locations, or that the method of material placement be observed and correlated with known results.

Where compaction measurement or control is desired or required, the recommended references are:

(a) ASTM D2049, "Standard Method for Test for Relative Density of Cohesionless Soils."
(b) ASTM D698, "Standard Method of Test for Moisture Density Relations of Soils Using 5.5-lb (2.5 kg) Rammer and 12-inch (204.8 mm) Drop."
(c) ASTM D2167, "Standard Method of Test for Density of Soil in Place by the Rubber-Balloon Method."
(d) ASTM D1556, "Standard Method of Test for Density of Soil in Place by the Sand-Cone Method."
(e) ASTM D2922, "Standard Method of Test of Density of Soil and Soil-Aggregate in Place by Nuclear Methods (Shallow Depth)."

It is recommended that the in-place density of Class I and Class II embedment materials be measured by ASTM D2049 by percentage of relative density, and Class III and Class IV measured by either ASTM D2167, D1556, or D2922, by percentage of Standard Proctor Density according to ASTM D698 or AASHTO T99.

F. Rigid Sewer Pipe Design

1. *General Relationships*

The inherent strength of a rigid sewer pipe usually is given by its strength in the three-edge bearing test. Although this test is both convenient and severe, it does not reproduce the actual field load conditions. Thus to select the most economical combination of bedding and sewer pipe strength, a relationship must be established between calculated load, laboratory strength, and field strength for various installation conditions (Figure 14.30).

Field strength, moreover, depends on the distribution of the reaction against the bottom of the sewer pipe and on the magnitude and distribution of the lateral pressure acting on the sides of the pipe. These factors, therefore, make it necessary to qualify the term "field strength" with a description of conditions of installation in a particular case, as they affect the distribution of the reaction and the magnitude and distribution of lateral pressure.

Just as for sewer pipe load computations, it is convenient when determining field strength to classify installation conditions as either trench or embankment.

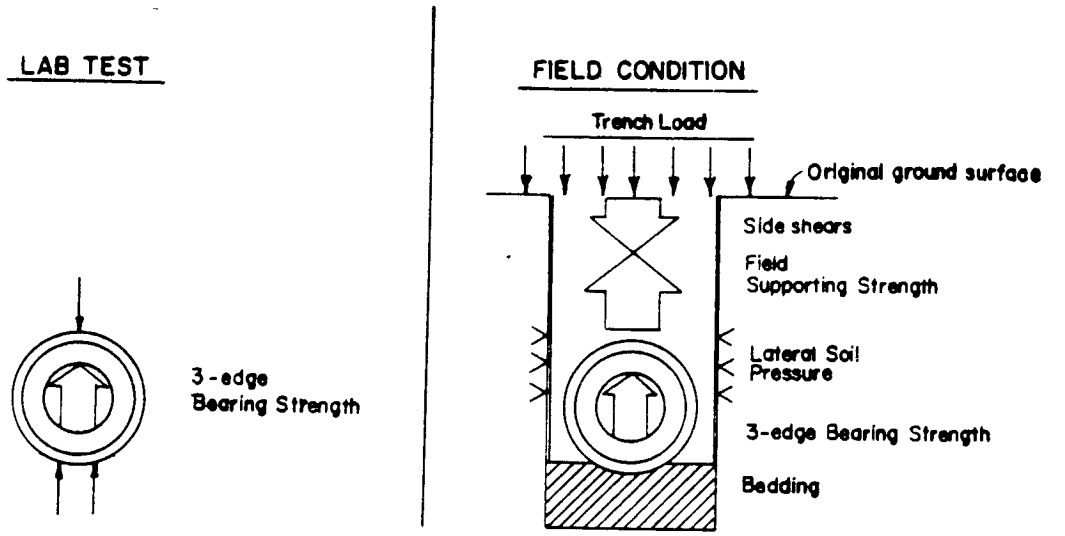

Figure 14.30—Both laboratory testing and field conditions should be used for rigid sewer pipe strength determination in the indirect design method (ASCE, 1982).

2. *Laboratory Strength*

Rigid sewer pipe is tested for strength in the laboratory by the three-edge bearing test. Methods of testing are described in detail in ASTM Specification C 301 for vitrified clay sewer pipe and C 497 for concrete and reinforced concrete sewer pipe.

The minimum strengths required for the three-edge bearing tests of the various types of rigid sewer pipe are stated in the ASTM Specifications for the pipe.

In the case of reinforced concrete sewer pipe, laboratory strengths are divided into two categories: The load that will produce a 0.01" crack, and the ultimate load the sewer pipe will withstand. The ultimate strength load in three-edge bearing is 1.50–1.25 times greater than the load that will produce a 0.01-in. crack, and the factor of safety used in design depends on which pipe class is being considered.

Non-reinforced concrete, and clay sewer pipe class designations indicate the ultimate strength in three-edge bearing directly in pounds per foot for any diameter.

3. *Design Relationships*

The structural design of rigid sewer pipe systems relates to the product's performance limit, expressed in terms of strength of the installed sewer pipe. Based on anticipated field loadings and concrete and reinforcing steel properties, calculations of shear, thrust, and moment can be made and compared to the ultimate strength of the section. This method is commonly used for reinforced concrete cast-in-place sewer pipes, and for precast concrete pipe using the direct design method.

For indirect design precast sewer pipes, the design strength is commonly related to a three-edge bearing test strength measured at the manufacturing plant, as described in the following text. The plant test is much more severe because maximum shearing forces occur at the point of maximum moment. Special reinforcement to resist shear in testing may be required by the more uniformly distributed field loading. In either case, the design load is equal to the field strength divided by the factor of safety. The design load is the calculated load on the pipe, and the field strength is the ultimate load the pipe must support when installed under specified conditions of bedding and backfilling.

The field strength is equal to the three-edge bearing test strength of the pipe times the bedding factor. The ratio of the strength of a sewer pipe under any stated condition of loading and bedding to its strength measured by the three-edge bearing test is called the bedding factor.

Bedding factors for trenches and embankments were determined experimentally between the years 1925 and 1935 at Iowa State College (Spangler 1956). The required three-edge bearing strength for a given class of bedding can be calculated as follows:

$$\text{Required Three-Edge Bearing Strength} = \frac{\text{Design Load} \times \text{Safety Factor}}{\text{Bedding Factor}}$$

The strength of reinforced concrete sewer pipe in pounds per foot at either the 0.01-inch crack or ultimate load, divided by the nominal internal diameter of the sewer pipe, is defined as the D-load strength: Different classes of reinforced concrete sewer pipe per ASTM C 76 and other reinforced concrete pipe specifications are defined by D-load strengths:

$$\text{D-Load} = \frac{\text{Design Load} \times \text{Safety Factor}}{\text{Bedding Factor} \times \text{Diameter}}$$

In considering the design requirements, the design engineer should evaluate the options of different types of rigid sewer pipe and different bedding classes, keeping in mind the differences in the relationship between test loadings and field loadings.

4. *Rigid Sewer Pipe Installation—Classes of Bedding and Bedding Factors for Trench Conditions*

Four classes of beddings (Figure 14.31) are used most often for sewer pipes in trenches (Moser et al. 1977; Sikora 1980). They are described in the following subsections.

Class A—Concrete Cradle The sewer pipe is bedded in a cast-in-place cradle of plain or reinforced concrete having a thickness equal to one-fourth the inside pipe diameter, with a minimum of 4 inches and a maximum of 15 inches under the pipe barrel and extending up the

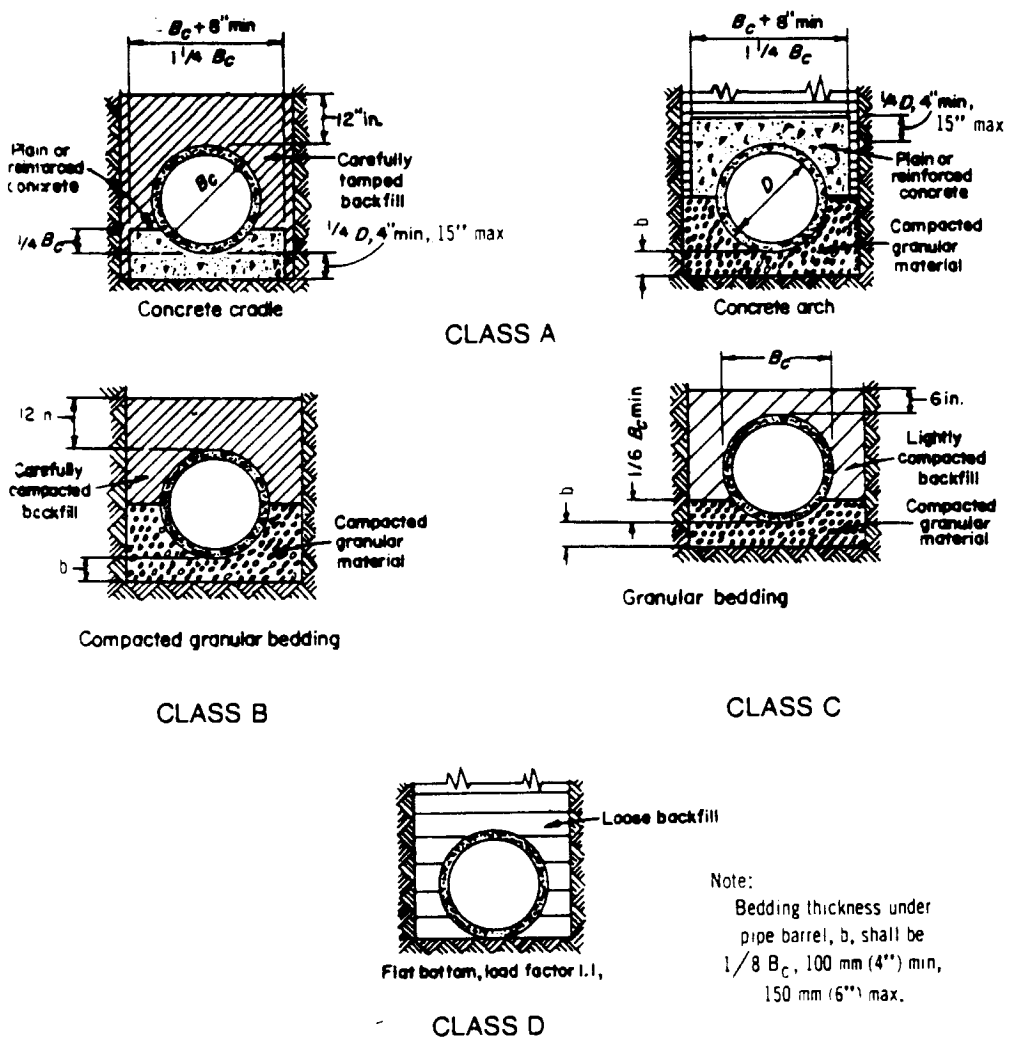

Figure 14.31—Classes of bedding for rigid sewer pipes in trench. NOTE: In rock trench, excavate at least 15 cm (6 in.) below bell of pipe except where concrete cradle is used (in. × 25.4 = mm) (ASCE, 1982).

sides for a distance equal to one-fourth the outside diameter. The cradle shall have a width at least equal to the outside diameter of the sewer pipe barrel plus 8 inches. Construction procedures must be executed carefully to prevent the sewer pipe from floating off line and grade during placement of the cradle concrete.

If the cradle is made of reinforced concrete, the reinforcement is placed transverse to the pipe and 3 inches clear from the bottom of the cradle. The percentage of reinforcement, p, is the ratio of the area of transverse reinforcement to the area of concrete cradle at the pipe invert above the centerline of the reinforcement.

Consideration must be given to the points at which the cradle (or arch, in the next section) begins and terminates with respect to the pipe

joints. In general, the concrete cradle, or envelope should start and terminate at the face of a pipe bell or collar to avoid shear cracks.

Haunching and initial backfill above the cradle to 12″ above the crown of the sewer pipe should be placed and compacted as described earlier in this chapter. The cradle must be cured sufficiently to develop full bedding prior to final backfilling.

The bedding factor for Class A concrete cradle bedding is 2.2 for plain concrete with lightly tamped backfill; 2.8 for plain concrete with carefully tamped backfill; up to 3.4 for reinforced concrete with the percentage of reinforcement, p, equal to 0.4%, and up to 4.8 with p equal to 1.0%.

Class A—Concrete Arch The sewer pipe is bedded in carefully compacted granular material having a minimum thickness of one-eighth the outside diameter but not less than 4″ or more than 6″ between the sewer pipe barrel and bottom of the trench excavation. Granular material is then placed to the spring line of the sewer pipe and across the full breadth of the trench. The haunching material beneath the sides of the arch must be compacted so as to be unyielding. Crushed stone in the 0.25–0.75″ size range is the preferred material. The top half of the sewer pipe is covered with a cast-in-place plain or reinforced concrete arch having a minimum thickness of 4″ or one-fourth the inside pipe diameter (but not to exceed 15 inches), and having a minimum width equal to the outside sewer pipe diameter plus 8″.

If the arch is made of reinforced concrete, the reinforcement is placed transverse to the pipe and 2″ clear from the top of the arch. The percentage of reinforcement is the ratio of the transverse reinforcement to the area of concrete arch above the top of the pipe and below the centerline of the reinforcement.

Class B Bedding The sewer pipe is bedded in carefully compacted granular material. The granular bedding has a minimum thickness of one-eighth the outside sewer pipe diameter, but not less than 4″ or more than 6″, between the barrel and the trench bottom, and covering the full width of the trench.

The haunch area of the sewer pipe must be fully supported; therefore, the granular material should be shovel sliced or otherwise compacted under the pipe haunch. Both haunching and initial backfill to a minimum depth of 12″ over the top of the sewer pipe should be placed and compacted.

The bedding factor for Class B bedding is 1.9.

Class C Bedding The sewer pipe is bedded on compacted granular material. The bedding has a minimum thickness of one-eighth the outside sewer pipe diameter, but not less than 4″ or more than 6″, and shall extend up the sides of the sewer pipe one-sixth of the pipe outside diameter. The remainder of the sidefills, to a minimum depth of 6″ over the top of the pipe, consists of lightly compacted backfill.

The bedding factor for Class C bedding is 1.5.

Class D Bedding For this class of bedding, the bottom of the trench is left flat, as cut by excavating equipment. Successful Class D installations of rigid conduits can be achieved in locations with appropriate soil conditions, trench width control, and light superimposed loads. Care must be taken to prevent point loading of sewer pipe bells; the excavation of bell holes will prevent such loading. Existing soil should be shovel sliced or otherwise compacted under the haunching of the sewer pipe to provide some uniform support. In poor soils, granular bedding material is generally a more practical and cost effective installation.

Since the field conditions with Class D bedding can approach the load conditions for the three-edge bearing test, the bedding factor for Class D beddings is 1.1.

5. *Variable Bedding Factors for Trench Conditions*

Both Spangler (1941) and Schlick (1932) postulate that some active lateral pressure is developed in trench installations before the transition width is reached. Experience indicates that the active lateral pressure increases as the trench width increases from a very narrow width to the transition width, provided the sidefill is compacted. Defining the narrow trench width as a trench having a width at the top of the pipe equal to or less than the outside horizontal span plus one foot, and assuming a conservative linear variation, the variable trench bedding factor can be determined by:

$$B_{fv} = (B_{fe} - B_{ft}) \left[\frac{B_d - (B_c + 1.0)}{B_{dt} - (B_c + 1.0)} \right] + B_{ft} \qquad (14\text{-}30)$$

in which B_c is the outside horizontal span of pipe, in feet; B_d is the trench width at top of pipe; B_{dt} is the transition width at top of pipe; B_{fe} is the bedding factor, embankment; B_{ft} is the fixed bedding factor, trench; and B_{fv} is the variable bedding factor, trench.

A six-step design procedure for determining the trench variable bedding factor is:

(a) Determine the trench fixed bedding factor, B_{ft}.
(b) Determine the trench width, B_d.
(c) Determine the transition width for the installation conditions, B_{dt}.
(d) Determine H/B_c ratio, settlement ratio, r_{sd}, projection ratio, p, and the product of the settlement and projection ratios, $r_{sd}p$.
(e) Determine positive projecting embankment bedding factor, B_{fe}.
(f) Calculate the trench variable bedding factor, B_{fv}.

6. *Encased Pipe*

Total encasement of rigid sewer pipe in concrete may be necessary where the required field strength cannot be obtained by other bedding or installation methods.

A typical concrete-encasement detail is shown in Figure 14.32. The bedding factor for concrete encasement varies with the thickness of concrete and the use of reinforcement, and may be greater than that for a concrete cradle or arch. Concrete thickness and reinforcement should be determined by the application of conventional structural theory and analysis. The bedding factor for the encasement shown in Figure 14.32 is 4.5.

Concrete encasement also may be required for sewers built in deep trenches to ensure uniform support, or for sewer pipe built on comparatively steep grades where there is the possibility that earth beddings may be eroded by currents of water under and around the pipe. Flotation of the pipe during concrete placement should be prevented.

7. *Field Strength in Embankments*

The active soil pressure against the sides of rigid sewer pipe placed in an embankment can be a significant factor in the resistance of the structure to vertical load. This factor is important enough to justify a separate examination of the field strength of embankment sewer pipe.

The following discussion of field strength in embankments is based on a theory developed by Marston and Spangler. The design engineer, however, should approach design based on this theory with some caution. With time, lateral pressures for trench installation usually will

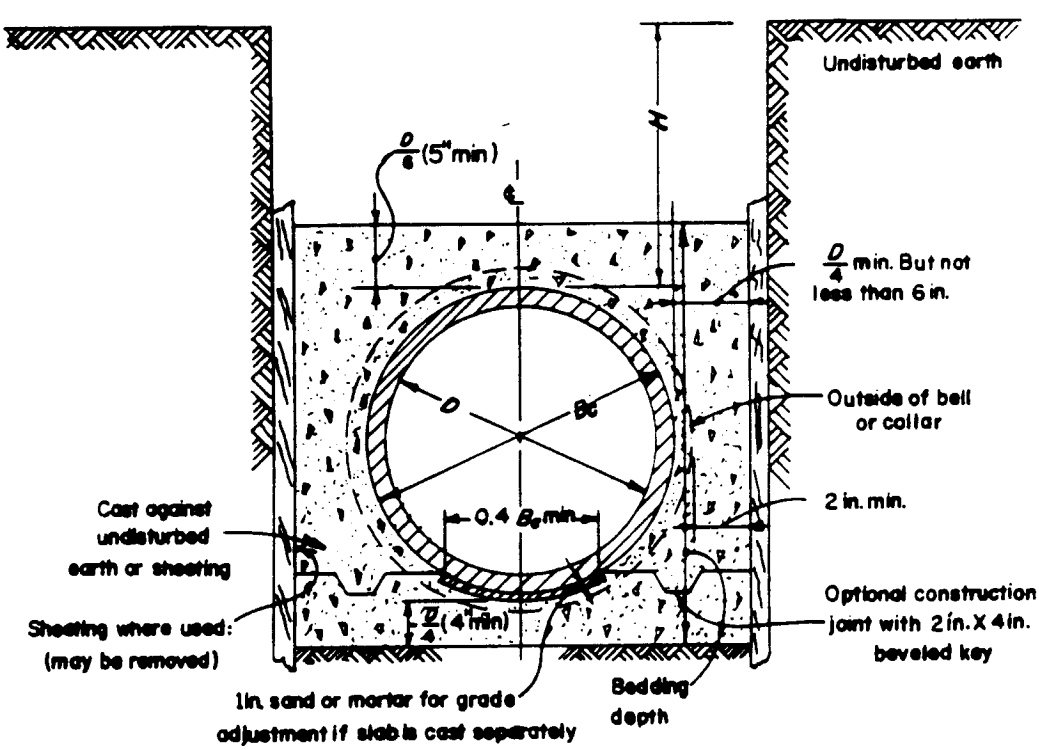

Figure 14.32—Typical concrete encasement details (in. × 25.4 = mm) (ASCE, 1982).

approach "at rest" conditions, which correspond to vertical overburden times a factor (usually between 0.5 and 1.0). However, for a negative projecting conduit, a positive projecting conduit, and induced trench embankment conditions, the lateral pressure magnitude and distribution may be much different, and these may control the structural design of the sewer pipe.

Positive Projecting Sewer Pipe The bedding factor for rigid sewer pipes installed as projecting sewer pipe under embankments or in wide trenches depends on the bedding in which the sewer pipe is laid, the magnitude of the active lateral soil pressure against the sides of the sewer pipe, and the area of the sewer pipe over which the active lateral pressure is effective.

For projecting sewer pipe, the bedding factor B_f, is:

$$B_f = \frac{A}{N - xq} \tag{14-31}$$

in which A is a sewer pipe shape factor; N is a parameter that is a function of the bedding class; x is a parameter dependent on the area over which lateral pressure effectively acts; and q is the ratio of total lateral pressure to total vertical load on the sewer pipe.

Classes of bedding for projecting sewer pipe are shown in Figure 14.33. The values of A for circular, elliptical, and arch sewer pipe are shown in Table 14.15.

Values of N for various classes of bedding are given in Table 14.16. Values of x for circular, elliptical and arch sewer pipe are listed in Table 14.17. The projection ratio, m, refers to the fraction of the sewer pipe diameter over which lateral pressure is effective. For example, if lateral pressure acts on the top half of the sewer pipe above the horizontal diameter, m equals 0.5. The ratio of total lateral pressure to total vertical load, q, for positive projecting sewer pipe may be estimated by the formula:

$$q = \frac{m\kappa}{C_c}\left(\frac{H}{B_c} - \frac{m}{2}\right) \tag{14-32}$$

in which κ is the ratio of unit lateral pressure to unit vertical pressure (Rankine's ratio). A value of κ equal to 0.33 usually will be sufficiently accurate for use in Equation (14-32). Values of C_c are found in Figure 14.10.

Negative Projecting Sewer Pipe The bedding factor for negative projecting sewer pipe may be the same as that for trench conditions corresponding to the various classes of bedding given in this chapter. The bedding factors for Class B, C, and D trench bedding do not take into account lateral pressures against the sides of the sewer pipe. However,

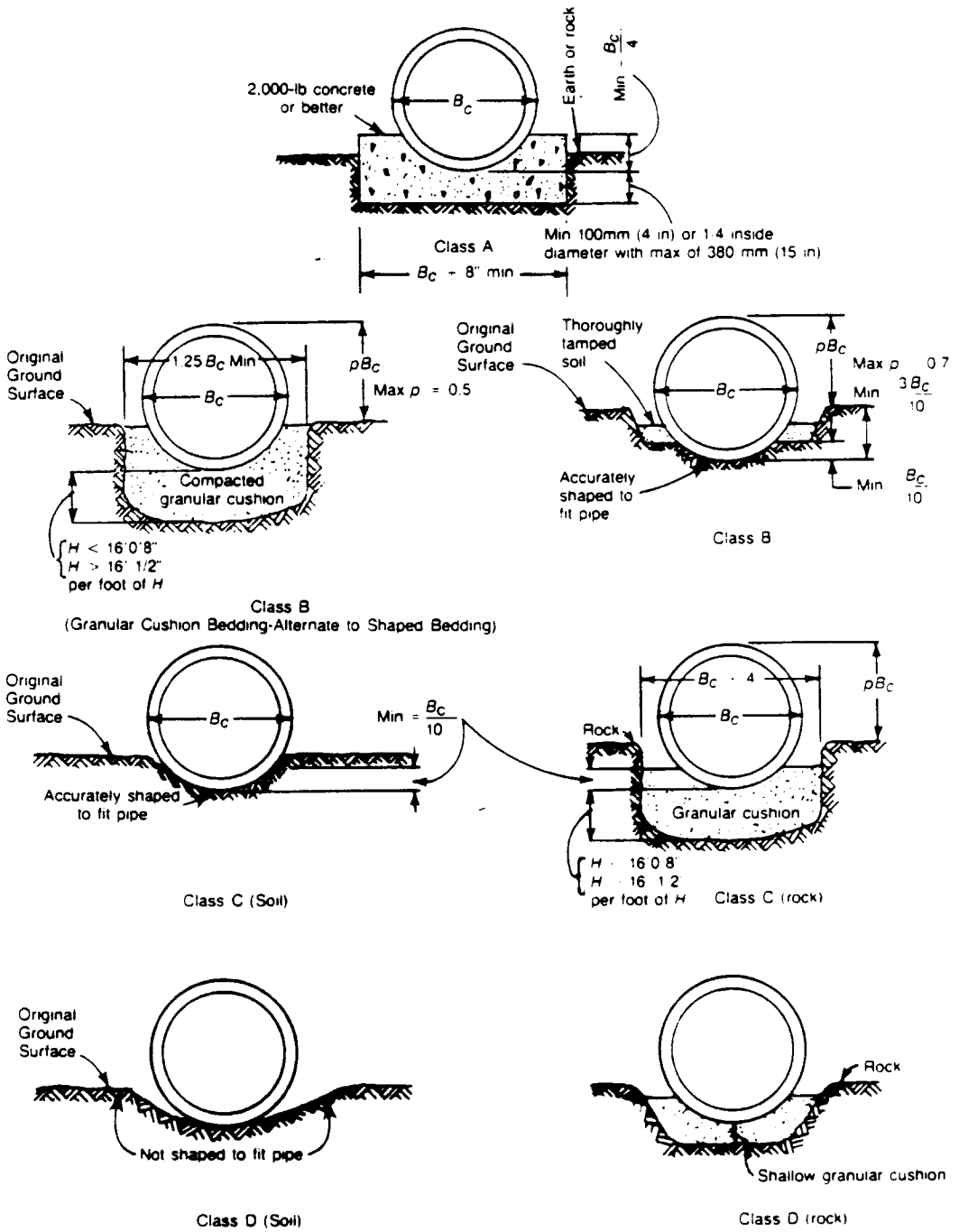

Figure 14.33—Classes of bedding for projecting sewer pipes (ft × 0.304 8 = m; in. × 25.4 = mm) (ASCE, 1982).

TABLE 14.15. Values of A for Circular, Elliptical, and Arch Sewer Pipe (ASCE 1982).

Sewer Pipe Shape (1)	A (2)
Circular	1.431
Elliptical:	
Horizontal elliptical and arch	1.337
Vertical elliptical	1.021

TABLE 14.16. Values of N (ASCE 1982).

	Value of N		
	Sewer Pipe Shape		
Class of Bedding (1)	Circular (2)	Horizontal Elliptical (3)	Vertical Elliptical (4)
A (reinforced cradle)	0.421 to 0.505	—	—
A (unreinforced cradle)	0.505 to 0.636	—	—
B	0.707	0.630	0.516
C	0.840	0.763	0.615
D	1.310	—	—

TABLE 14.17. Values of x (ASCE 1982).

	x			
	Class A Bedding	Other than Class A Bedding		
Fraction of Sewer Pipe Subjected to Lateral Pressure, m (1)	Circular Sewer Pipe (2)	Circular Sewer Pipe (3)	Horizontal Elliptical Sewer Pipe (4)	Vertical Elliptical Sewer Pipe (5)
0	0.150	0	0	0
0.3	0.743	0.217	0.146	0.238
0.5	0.856	0.423	0.268	0.457
0.7	0.811	0.594	0.369	0.639
0.9	0.678	0.655	0.421	0.718
1.0	0.638	0.638	—	—

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in the case of negative projecting sewer pipe, it may be possible to compact the sidefill soils to the extent that some lateral pressure against the sewer pipe can be relied on. If such favorable conditions are anticipated, it is suggested that the bedding factor be computed by means of Equations (14-31) and (14-32), using a value of κ equal to 0.15 for estimating the lateral pressure on the sewer pipe.

Induced Trench Conditions Induced trench sewer pipes usually are installed as positive projecting sewer pipes before the overlying soil is compacted and the induced trench is excavated. Therefore, lateral pressures are effective against the sides of the sewer pipe, and the bedding factor should be calculated using Equations (14-31) and (14-32).

G. Flexible Sewer Pipe Design

Several types of flexible pipe are available for use as sewer pipe material. Among the most common are coated corrugated metal pipe (CMP), acrylonitrile-butadiene-styrene (ABS) composite pipe and (ABS) solid wall pipe, polyvinyl chloride (PVC) pipe, polyethylene (PE) pipe, and fiberglass-reinforced plastics (FRP).

1. General Method

Flexible sewer pipes under earth fills and in trenches derive their ability to support load from their inherent strength plus the passive resistance of the soil as the pipe deflects and the sides of sewer pipe move outward against the soil sidefills. Proper compaction of the soil sidefills is important to the long term structural performance of flexible sewer pipe. The extent to which flexible pipe deflects is most commonly used to judge performance and as a basis for design. The amount of deflection considered permissible is dependent on physical properties of the pipe material used and project limitations.

The limiting buckling stress for flexible pipes takes into account the restraining effect of the soil structure around the pipe and the properties of the pipe wall. Equations for the critical stress in the pipe wall can be found in manufacturers' handbooks for the various types of pipe.

The approximate long-term deflection of flexible sewer pipe can be calculated using the Modified Iowa Formula developed by Spangler and Watkins, provided the pipe to soil stiffness relationship produces an elliptical deformation:

$$\Delta X = \frac{D_t K_b W_e r^3}{EI + 0.061\, E' r^3} \tag{14-33}$$

where

ΔX = horizontal deflection, in inches;
K_b = bedding factor;

D_L = deflection lag factor;
W_e = load, in pounds per linear inch;
r = mean radius of pipe, in inches;
E = modulus of tensile elasticity, in pounds per square inch;
I = moment of inertia per length, in inches to the fourth power per inch; and
E' = modulus of soil reaction, in pounds per square inch.

For small deflections, the vertical deflection ΔY may be assumed to approximately equal the horizontal deflection ΔX in Equation (14-33). Research has been conducted on the application of Equation (14-33) to pipe having a low ratio of pipe stiffness to soil stiffness where the vertical deflection may not be assumed to equal the horizontal deflection. For low pipe to soil stiffness ratios, a correction factor must be applied to the calculated horizontal deflection to accurately predict the vertical deflection (Zicaro 1990).

Flexible sewer pipes that are to support a fill should not be placed directly on a cradle or pile bents. If such supports are necessary, they should have a flat top and be covered with a compressible earth cushion. In those instances where flexible pipe is to be encased in concrete, the pipe manufacturer should be consulted.

The deflection lag factor, empirically determined, compensates for the time consolidation characteristics of the soil, which may deform flexible sewer pipes for some period after installation. Long-term deflection will be greater with light or moderate degrees of compaction of sidefills when compared to values for heavy compaction. The better the compaction, the lower the initial deflection, and the greater the magnitude of the long-term lag factor. Lag factors over 2.5 have been recorded in dry soil. Recommended values of this factor range from 1.25–2.50. Bedding requirements for flexible sewer pipe installations are discussed below.

Values of the bedding constant K_b, depending on the width of the sewer pipe bedding, are shown in Table 14.18.

TABLE 14.18. Values of Bedding Constant, K_b (ASCE 1982).

Bedding Angle, in degrees (1)	K_b (2)
0	0.110
30	0.108
45	0.105
60	0.102
90	0.096
120	0.090
180	0.083

Note: 1 deg = 0.017 rad.

The passive resistance of the soil at the sides of the pipe greatly influences flexible pipe deflection. This passive resistance is expressed as the Modulus of Soil Reaction, E'. It is an empirical number related to the degree of compaction of the soil and to the type of soil. The U.S. Bureau of Reclamation (Howard 1977) has established an empirical relationship between Modulus of Soil Reaction values, degree of compaction of bedding, and type of bedding material. These values are shown in Table 14.19. (See Table 14.14 for soil symbol definition.)

In the deflection formula (Equation (14-33)) the first term in the denominator, EI (stiffness factor), reflects the influence of the inherent stiffness of the sewer pipe on deflection. The second term, $0.061\ E'r^3$, reflects the influence of the passive pressure on the sides of the pipe. The second term may predominate in the case of large-diameter pipes, with the result that a very lightweight pipe may appear to be satisfactory. Since the pipe wall must have sufficient local strength in bending and thrust to develop and utilize the passive resistance pressure on the sides of the pipe, it is recommended as a practical measure that the value of EI should never be less than about 10–15% of the term $0.061\ E'r^3$.

It should be noted that the E' values in Table 14.19 are average values and do not reflect field variables. A conservative approach would be to use 75% of the E' values given to calculate maximum deflections.

The ability of a flexible sewer pipe to retain its shape and integrity is largely dependent on the selection, placement, and compaction of the envelope of soil surrounding the structure. For this reason as much care should be taken in the design of the bedding and initial backfill as is used in the design of the sewer pipe. The backfill material selected preferably should be of a granular nature to provide good shear characteristics. Cohesive soils are generally less suitable because of the importance of proper moisture content and the difficulty of obtaining proper compaction in a limited work space.

If, under embankment conditions, the material placed around the sewer pipe is different from that used in the embankment, or if for construction reasons fill is placed around the sewer pipe before the embankment is built, the compacted backfill should cover the pipe by at least 1 foot.

Design Relationships Structural design of flexible pipe requires definition of the critical deflection limit for the specific pipe considered. The critical deflection limit for flexible pipe is commonly based on structural performance characteristics and potential modes of failure. The requirements of serviceability, such as effect on carrying capacity, passage of cleaning equipment, velocity and differential deflection at fittings, manholes and joints, should be considered. Maximum long-term strain in the pipe wall, discussed above, also should be considered. The design basis is expressed as follows:

TABLE 14.19. Bureau of Reclamation Average Values of E' for Iowa Formula (for Initial Flexible Pipe Deflection) (ASCE 1982).

Soil Type-Pipe Bedding Material (Unified Classification System[a]) (1)	E' for Degree of Compaction of Bedding, in pounds per square inch			
	Dumped (2)	Slight, <85%, Proctor, <40% Relative Density (3)	Moderate, 85%–95% Proctor, 40%–70% Relative Density (4)	High >95% Proctor, >70% Relative Density (5)
Fine-grained Soils (LL > 50)[b] Soils with medium to high plasticity CH, MH, CH-MH	No data available; consult a competent soils engineer: Otherwise use $E' = 0$			
Fine-grained Soils (LL < 50) Soils with medium to no plasticity CL, ML, ML-CL, with less than 25% coarse-grained particles	50	200	400	1,000
Fine-grained Soils (LL < 50) Soils with medium to no plasticity CL, ML, ML-CL, with more than 25% coarse-grained particles Coarse-grained Soils with Fines GM, GC, SM, SC[c] containes more than 12% fines	100	400	1,000	2,000
Coarse-grained Soils with Little or No Fines GW, GP,SW,SP[c] contains less than 12% fines	200	1,000	2,000	3,000
Crushed Rock	1,000	3,000	3,000	3,000
Accuracy in Terms of Percentage Deflection[d]	±2	±2	±1	±0.5

[a]ASTM Designation D-2487, USBR Designation E-3
[b]LL = Liquid limit.
[c]Or any borderline soil beginning with one of these symbols (i.e., GM-GC, GC-SC).
[d]For ±1% accuracy and predicted deflection of 3%, actual deflection would be between 2% and 4%.
Note: Values applicable only for fills less than 50 ft (15 m). Table does not include any safety factor. For use in predicting initial deflections only, appropriate Deflection Lag Factor must be applied for long-term deflections. If bedding falls on the borderline between two compaction categories, select lower E' value or average the two values. Percentage Proctor based on laboratory maximum dry density from test standards using about 12,500 ft-lb/cu ft (598,000 J/m^3) (ASTM D-698, AASHO T-99, USBR Designation E-11). 1 psi = 6.9 KPa.

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$$\text{Design Deflection Limit} = \frac{\text{Allowable Deflection Limit}}{\text{Factor of Safety}}$$

In structural design for a flexible sewer pipe system, the design deflection limit should not be more than the maximum long-term deflection anticipated under load. In the determination of projected long-term deflection, the design element considerations are load (soil, superimposed dead load, and superimposed live load), pipe stiffness and soil stiffness. The primary controlling elements of deflection are the load, the pipe stiffness, the soil stiffness and the ratio of pipe to soil stiffness. Typical allowable long-term deflection limits are:

- Factory cement-mortar-lined and/or cement-mortar-coated steel pipe 3%
- Flexible lined and coated steel pipe 5%
- Flexible coated and cement-mortar-lined in-place steel pipe 5%
- Fiberglass pressure pipe 5%
- Thermoplastic pipe (PVC) 5%

The soil stiffness depends on the type of soil and its density as placed in the sewer pipe embedment zone. In consideration of design requirements, the engineer should evaluate the options of different types of flexible sewer pipes, different embedment soils, and different embedment material compaction and placement requirements.

The choice of factor of safety should be influenced by soil characteristics, the degree of compaction likely to be obtained, and available field tests, and practical experience.

Loads on Flexible Pipe The load carried by a buried flexible pipe in a narrow trench may be calculated using the Marston Formula, Equation (14-13). A conservative design approach may be used by assuming the dead load carried by a flexible pipe-soil system in any installation to be the prism load. For normal installations the prism load is the maximum load that can be developed.

The load on a projecting flexible sewer pipe is calculated using Equation (14-14). As before, the load coefficient (C_c) depends on the projection ratio (p), settlement ratio r_{sd}, and the ratio of fill height to pipe width (H/B_c), and can be determined from Figure 14.10. For flexible projecting pipe, the product $r_{sd}p$ is negative or zero. As shown in Figure 14.10, when the product is zero, the load coefficient, C_c, equals H/B_c. Equation (14-14) then becomes Equation (14-16), which gives the prism load.

2. *Design of Plastic Sewer Pipe*

Thermoplastic pipe materials (ABS, PE, and PVC) are all affected by temperature. Tests are usually specified to be conducted at 23° C (73.4° F). At higher temperatures the pipe stiffness is decreased, and lower temperatures result in greater pipe stiffness. Pipe being installed at high and low temperatures requires careful handling.

Laboratory Load Test The standard test to determine "pipe stiffness" or load deflection characteristics of plastic pipe is the parallel-plate loading test. The test is conducted in accordance with ASTM C 2412, "Standard Test Method for External Loading Properties of Plastic Pipe by Parallel-Plate Loading."

In the test, a short length of pipe is loaded between two rigid parallel flat plates that are moved together at a controlled rate. Load and deflection are noted.

The parallel-plate loading test determines the pipe stiffness (PS) at a prescribed deflection (ΔY), which for convenience in testing is arbitrarily set at 5%. This is not to be considered the field deflection limitation. The pipe stiffness is defined as the value obtained by dividing the force (F) per unit length by the resulting deflection in the same units at the prescribed percentage deflection, and is expressed in pounds per inch:

$$PS = \frac{F}{\Delta Y} = \frac{EI}{0.149r^3} \tag{14-34}$$

where

E = modulus of elasticity in pounds per square foot;
I = $t^3/12$;
r = mean radius of pipe; and
t = wall thickness.

Minimum required pipe stiffness values are stated in plastic sewer pipe specifications. Table 14.20 lists the ASTM Specifications for various types of plastic pipe and the corresponding pipe stiffness values.

The stiffness factor (SF) is the pipe stiffness multiplied by the quantity $0.149r^3$:

$$SF = EI = \frac{F}{\Delta Y} 0.149r^3 = 0.149r^3\,(PS) \tag{14-35}$$

The stiffness factor or EI is used in the Modified Iowa Formula (Equation 14-33) to determine approximate field deflections under earth loads. It is the engineer's responsibility to establish the acceptable field deflection limit, and to design the installation accordingly. The manufacturer should be consulted for recommended field installation deflection limits.

Field Deflection of Flexible Plastic Pipe As previously discussed, the pipe stiffness is determined in a standard parallel-plate loading test for an arbitrary deflection of the nominal diameter. This is a measure of the inherent strength of plastic pipe.

The pipe stiffness and soil stiffness as measured by E' must develop sufficient field strength that the deflection of the pipe under load will not exceed the acceptable deflection.

TABLE 14.20. Stiffness Requirements for Plastic Sewer Pipe Parallel-Plate Loading* (ASCE 1982).

Material** (1)	ASTM Specification (2)	Nominal Diameter, d, in inches (3)	Required Stiffness at 5% Deflection, in pounds per square inch (4)
ABS Composite	D 2680	8–15	200
ABS Plain	D 2751		
	SDR 23.5	4 & 6	150
	SDR 35	3	50
		4 & 6	45
	SDR 42	8, 10 & 12	20
RPM	D 3262	8–18	Varies (99–17)
		20–108	10
PVC	D 2729	2	59
	(PVC-12454)	3	19
		4	11
		5	9
		6	8
	D 2729	2	74
	(PVC-13364)	3	24
		4	13
		5	12
		6	10
	D 3033		
	SDR 41	6–15	28
	SDR 35	4–15	46
	D 3034		
	SDR 41	6–15	28
	SDR 35	4–15	46

*ASTM 2412
**Other plastic pipe materials are not listed, in that insufficient data is currently available.
Note: 1 in. = 25.4 mm; 1 psi = 6.89 kPa.

Approximate values of horizontal deflections for field installations can be calculated using the Modified Iowa Formula (Equation (14-33)). A correction factor must be applied to the calculated horizontal deflection to accurately predict the vertical deflection for low pipe to soil stiffness ratios (Zicaro 1990). This formula can be simplified to permit a calculation of approximate deflection based on pipe stiffness as follows:

$$\Delta X = \frac{D_L K_b W_c}{0.149PS + 0.061E'} \tag{14-36}$$

Values of the bedding constant for use in this formula were presented earlier in the "General Methods" subsection of this section.

The solution of Equations (14-33) and (14-36) requires that various factors be determined. It is desirable, where possible, to establish anticipated long-term field deflections based on well-documented empirical data.

3. *Soil Classification*

For different categories of embedment materials, different construction procedures are specified. Soil classifications under the Unified Soil Classification System, including manufactured materials, are grouped into five broad categories according to their ability to develop an interacting sewer pipe-soil system (Table 14.14).

For the larger-grained soils (Class I and Class II gravels), compatibility with the existing subgrade and trench side soils should be considered. Particularly for uniformly graded or gap graded materials, the potential exists for the migration of the finer fraction of the existing soils into the embedment materials, with resultant settlement and loss of side support for the sewer pipe. Analytical techniques for assessing this possibility are similar to those for filter blankets and are well covered in the references. See Table 14.19 for the suggested E' values given for each soil class.

Class I This class includes angular, 0.25–1", graded stone, including a number of fill materials that have regional significance, such as coral, clay, cinders, crushed stone, and crushed shells.

Class I material provides the best material for the construction of a stable sewer pipe-soil system. When used for underdraining, Class I material should be placed to the top of the sewer pipe.

Class I material used for haunching and initial backfill, when dumped into place with little or no compaction, will produce an average E' value of 1,000 psi. Care must be taken to place material under the haunches and in contact with the sides of the pipe. Class I material compacted to 85% Standard Proctor Density or higher has an average E' value of 3,000 psi.

Class II This class comprises coarse sands and gravels with maximum particle size of 1.5", including variously graded sands and gravels containing small percentages of fines, generally granular and non-cohesive, either wet or dry. Soil types GW, GP, SW, and SP are included.

Use of sands of Type SP should be done with caution. Poorly graded fine sands with little material finer than that passing a 200-sieve have a tendency to flow when wet.

Class II material used for haunching and initial backfill, when dumped into place with little or no compaction, will product an average E' value of 200 psi. Compacted to 85% Standard Proctor Density, the average E' value is raised to 1,000 psi. Higher E' values are obtained with greater compaction.

Class III This class comprises fine sand and clayey gravels, including fine sands, sand-clay mixtures, and gravel-clay mixtures. Soil types GM, GC, SM and SC are included.

Class III materials dumped in place for haunching and initial backfill will produce an average E' of 100 psi. Careful placement and compaction to 90% Standard Proctor Density will produce an average E' of 1,000 psi.

Class IV and V Class IV materials require special effort for compaction, thus may be suitable for sewer pipe foundation if special care is taken during excavation to provide a uniform, undisturbed trench bottom. Use of Class IV materials for bedding, haunching, or initial backfilling is not recommended.

Class V materials present special problems in providing an adequate foundation, and should not be used for any part of the sewer pipe envelope.

4. *Design of Corrugated Metal Sewer Pipes*

Corrugated metal pipe is manufactured in a variety of gages, corrugation depths, and corrugation spacings.

The longitudinal seam formed by bolting or riveting curved sheets together for corrugated metal pipe should be checked for crushing strength. Tables of seam strengths for various metal gages and bolt or rivet sizes and spacing can be found in manufacturers' handbooks. To improve hydraulic properties, the pipe may have a paved invert (25% of circumference) or may have the interior completely paved.

Corrugated metal pipe sewers may be designed for a limiting deflection using the Modified Iowa Formula (Equation (14-33)) or by the manufacturer's handbook figures. A design maximum deflection limitation of 5% is commonly used.

Corrugated metal sewers that are to support a fill should not be placed directly on a cradle or pile bents. If such supports are necessary, they should have a flat top and be covered with a compressible earth cushion. Corrugated metal should not be encased in concrete. For corrugated metal pipes installed in trenches, reference is made to manufacturers' handbooks for recommended gages and corrugations.

5. *Flexible Sewer Pipe Installation*

General bedding and backfilling concepts developed for rigid sewer pipe may also be applied to flexible pipe, with some exceptions. The general pipe terminology for installation was shown in Figure 14.29

Detailed information on bedding of flexible pipe is contained in the various ANSI and ASTM specifications or manufacturers' literature. The design engineer should consult the applicable specification or literature for design information.

Bedding, Haunching and Initial Backfill The bedding requirements for flexible thermoplastic sewer pipe are given in ASTM D 2321 for

Class I, II, and III material (Table 14.14). Haunching and initial backfill requirements are also given, including minimum compaction recommendations. Similarly, bedding and initial backfill requirements for thermosetting reinforced plastic pipe are given in ASTM D 3839, and a guide for estimating both initial and long-term deflection is included in the appendices. Installation requirements for corrugated metal pipe are given in ASTM A 798, which describes procedures, soils and soil placement.

By referring to Tables 14.18 and 14.19, it is possible to estimate the E' value obtained for the expected compaction, or to determine the compaction requirements needed to develop the E' value that will keep the pipe deflection within allowable limits.

Final Backfill The procedure for installing final backfill for the remainder of the trench is the same as for rigid pipe.

IX. DESIGN OF OTHER SPECIFIC STRUCTURES

A. Open Channel Linings

Open channels are widely used in drainage and flood control projects, and often require linings that will remain stable during the flows the channels are designed to carry. Channel linings include bare earth, various forms of grasses and plants, riprap, grouted riprap, gabions, soil cement and concrete. Except for grouted rock, all were discussed in detail in Chapter 9.

1. Grouted Rock

Riprap channel linings can be grouted together to form a more rigid surface. The grouted riprap requires a lesser thickness of rock and reduces maintenance. However, the grouting results in the loss of several of the advantages of loose riprap. The lining's impermeability prevents groundwater pressures from being released between the rocks. The flow then concentrates at openings through the lining, with the potential of erosion of the underlying soil. The rigidity of the lining makes it more susceptible to failure if it is undermined by erosion or piping.

Grouted riprap channel linings form retaining walls and are loaded by lateral earth forces and groundwater pressures. As with riprap linings, a geotechnical engineer should be consulted regarding stable slopes which will not load the walls excessively.

2. Concrete

Channel linings are frequently constructed of reinforced concrete. The steeper the side slopes, the more the wall serves as a retaining wall. Reinforced concrete retaining walls can serve as channel lining in

two different structural configurations—as independent retaining walls or attached to a floor slab. These typical designs are illustrated in Figure 14.34.

Each wall can be an independent cantilever retaining wall with its own footing. In such a case, the channel bottom would not be connected to the walls, and thus could be concrete, riprap, or earth. Each wall would be structurally designed to resist lateral earth and groundwater pressures, and would require analyses of stability against overturning

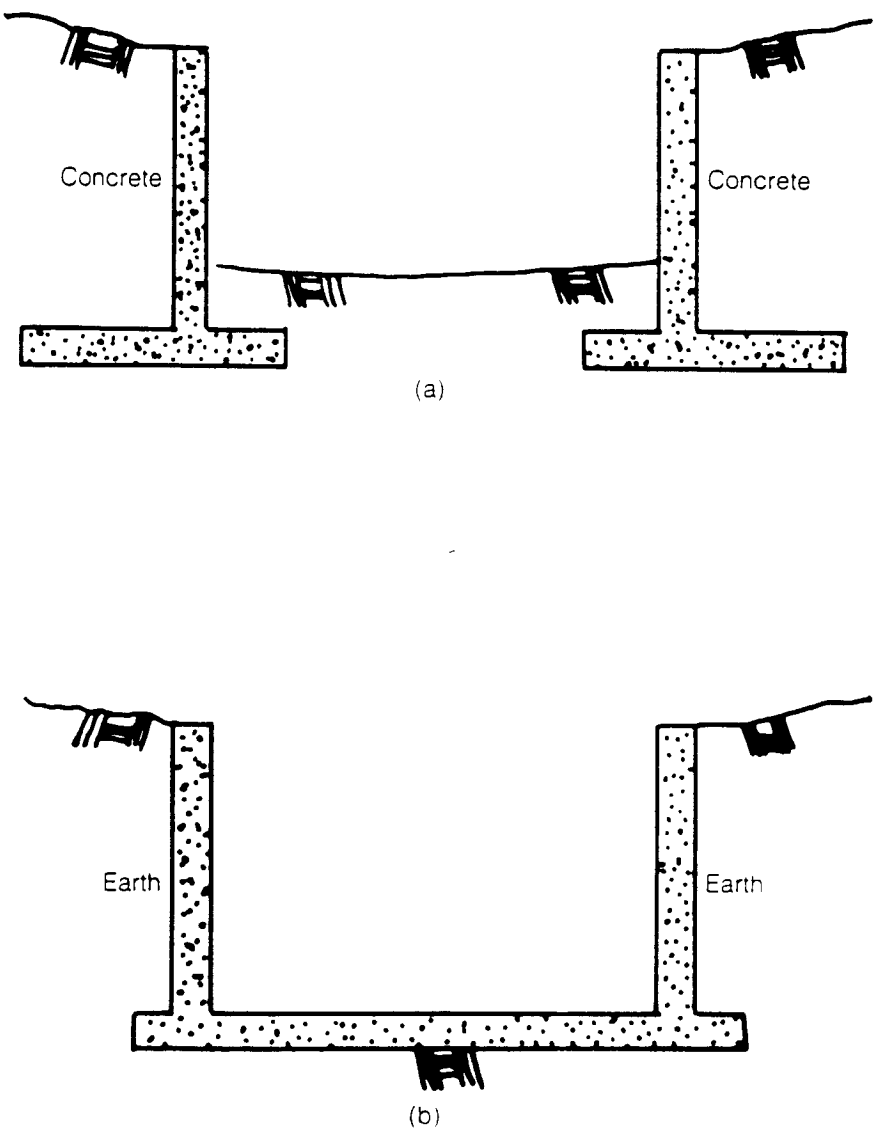

Figure 14.34—Alternate approaches to channel design with vertical concrete retaining walls (a) cantilever retaining walls with unlined channel bottom and (b) U-shaped channel with walls structurally connected to reinforced concrete floor.

and sliding, but would be little affected by uplift (except as it reduces the weight and thus friction resistance).

Reinforced concrete retaining walls can also be rigidly attached to the channel floor to form a rectangular or U-shaped channel. In this configuration, the same lateral soil and groundwater pressures are exerted on the walls, but the structure is subjected to little or no net overturning or sliding forces. The bending moment from the walls is transferred to the floor, which must also be a structural member. This increases the bending in the floor slab. These channels are subject to uplift forces and frequently require special attention to insure that flotation will not occur.

3. *Soil Cement and Roller Compacted Concrete*

These two materials are used for channel linings primarily to prevent bank erosion. They are generally designed to perform as gravity retaining walls to resist the lateral earth and groundwater pressures. The structures cannot be loaded in tension or bending, since the tensile strength, particularly in the vertical direction, is negligible. In a retaining wall configuration, the structures are subject to overturning and sliding (but have ample weight to resist both) in addition to uplift. Advantages of these products include relatively lower cost than concrete and a more attractive appearance (blends with natural soils).

4. *Geotechnical Considerations for Channel Linings*

For channels with vertical or near-vertical sides, the principal geotechnical considerations are stability, seepage control, and toe protection. For this type of channel, the side walls act as retaining structures, whether they are concrete walls, gabions, grouted riprap, or other types of materials, and should be evaluated as retaining walls. Evaluations of stability should include overturning and sliding. In addition, an evaluation should be made of the possibility of slope failure on a surface that passes completely beneath the structure. As for most retaining wall design, it is desirable to include seepage control measures to lower the water level behind the wall. In designing the seepage control measures, the engineer must provide adequate filters so that the facilities operate as intended. For a weep hole through a concrete wall, a two-stage filter consisting of concrete sand and a filter-compatible crushed stone should be used to keep the weep hole from plugging. Enough pervious soil must be placed behind the wall to allow the water to drain to the weep holes. All structural walls should be adequately embedded into the bottom of the channel, so that they are not undermined by scour or erosion.

For channels with steeply sloping sides requiring slope protection, the principal geotechnical considerations are slope stability, seepage control, adequate filter systems and toe protection. An analysis of the stability of the slope, including the protection system, should be made and the design slope angle selected accordingly. Seepage control sys-

tems should be included to keep water levels within limits assumed in the stability analyses. In addition, for impervious forms of slope protection, such as concrete, fabriform and grouted riprap, the seepage control systems must be sufficient to prevent build-up of enough water pressure beneath the protection to lift and damage it. Any open protection, such as riprap or gabions, should be underlain by filter layers, which prevent the underlying soils from being washed out either by groundwater or channel flow. For example, riprap should be underlain by successive layers of graded filters, such that the bottom layer is filter-compatible with the soils in the slope. The slope protection on steep slopes should be extended into the channel bottom so that it will not be undermined by erosion.

B. Open Channel Structures

The design of open channels usually requires the additional design of an assortment of associated structures, whose hydraulic functions and designs are described in Chapter 9. Following are structural comments on some of the more common open channel facilities.

1. Drop Structures

A drop structure is constructed to lower a channel abruptly in elevation. It usually serves as a combination of an energy dissipator and a retaining wall, and is commonly designed either as a cantilever retaining wall or a gravity retaining wall. Each structure must resist sliding in the direction of flow and overturning about the toe of the footing.

The design of the structure must also address the potential for development of groundwater pressures under the footing, with the resulting decrease in frictional resistance to sliding.

When a drop structure fails, it is frequently due to unexpected erosion at the toe or ends of the structure. The formation of a scour hole beneath the toe not only reduces the resistance to overturning and sliding, but also shortens the moment arm to the point about which the overturning could occur. For these reasons, it is important that riprap or other erosion protection be installed at the downstream edge of the structure.

2. Check Structures

A check structure differs from a drop structure in that there is no abrupt drop in the channel invert. A check structure forces the water upstream to flow at a higher elevation than would otherwise occur. In addition to hydraulic turbulence, the principal forces on a check structure are the hydrostatic and hydrodynamic forces of the water. The structure must be anchored in the earth by a foundation and must be evaluated for stability against sliding and overturning.

3. Energy Dissipators

An energy dissipator is constructed where the energy in flowing water must be reduced. The structural elements that create the reduc-

tion in energy may be integral with a conveying facility, such as a pipe or channel, or may be elements of a separate structure between two conveying facilities. Energy dissipators are frequently located at the bases of spillways, the ends of steep channels, or culvert outlets. The dissipation of energy generally results in complex hydraulic flow patterns that require simplification in order to compute the hydrostatic and hydrodynamic forces. The mechanism by which the energy is dissipated determines the manner in which the hydrodynamic forces of the water are transferred to the structure walls and interior elements, such as baffles.

The hydrodynamic loads tend to force the structure to slide downstream and to overturn about the toe. Stability against sliding and overturning must be evaluated. The turbulent flow in and around an energy dissipator can result in erosion of earthen areas. Care must be taken to insure that erosion is either prevented or is not critical to the structure's stability. Many energy dissipators are constructed with rigid floors and partly enclosed walls. In such cases, an evaluation of uplift is required.

C. Closed Conduit Appurtenances

Storm sewer systems and appurtenances represent a significant proportion of drainage and flood control structures. The appurtenant facilities that merit structural consideration include manholes, junction vaults, and inlets. Each is discussed briefly below:

1. Manholes

Under most circumstances, manholes present no stability concerns, because they are completely surrounded by soil. However, manholes are frequently subjected to significant superimposed loads from vehicular traffic. Manholes are designed in two different manners to resist the surface loads.

In one approach, a concrete footing is constructed around the manhole ring and cover, so that the vehicle load is immediately transferred to the soil adjacent to the manhole. This requires the ring and cover to be disconnected from the manhole ring, so that the load is not transmitted downward to the storm sewer.

In the second approach, the manhole ring is connected directly to the manhole cone. In this configuration, a vehicle's load is transmitted directly to the base of the manhole, with a small (and usually neglected) resistance provided by the soil friction on the outside of the manhole. If the manhole has a horizontal base, the load is transferred past the pipe to the earth, provided the manhole does not bear directly on the pipe. If the manhole is a "tee" manhole, directly connected to the pipe, or if it sits on a large storm sewer (either circular or rectangular in section), then the load is transferred directly to the pipe. The tee manhole connection makes the pipe inherently stronger, because of the stiffening of the structure at the opening. In addition, the bedding and

soil support under the pipe section must spread the load sufficiently so that the soil bearing pressure is not exceeded.

Differential settlement between the manhole and pipe may sometimes occur because of differences in bedding, vertical load and backfill settlement. If the pipe is rigidly connected to the manhole, differential settlement can induce excessive shear forces, resulting in circumferential cracks in the pipe, or shear failure. Resilient connectors accommodate differential settlements and minimize the effect of shear forces, as well as minimizing leakage by providing a positive seal between the manhole wall and the connector, and between the connector and the pipe. Resilient connectors are available throughout North America, and are manufactured in accordance with ASTM C 923.

2. *Junction Boxes*

Junction boxes or vaults are constructed in storm sewer systems when large pipes converge. Such vaults are subjected to the same concentrated loads as manholes described previously. In addition, the vaults experience lateral hydrodynamic forces if the flow changes direction or velocity. Under most circumstances, a lateral force in a storm sewer is easily resisted by the soil outside the storm sewer. However, small openings in pipe joints or between the vault and the pipe can allow jets of water to erode the soil outside of the pipe. The erosion, coupled with the hydrodynamic forces, can cause shifting of pipes, increased erosion, and eventual collapse of the backfill or the pipe itself.

Designers must insure that pipes adjacent to junctions have tight, impermeable joints and that the trench bedding and backfill are firmly compacted. In cases where lateral flows have high velocity, the designer should determine whether or not the lateral force would exceed the allowable passive resistance of the soil.

3. *Inlets*

Storm sewer inlets are generally simple structures but are subjected to numerous superimposed loads from motor vehicles. Any inlet, even if constructed primarily behind a curb line, can be crossed by wheels of heavily loaded trucks. The exposed parts of the inlet can also experience severe impact loads. The engineer cannot design an inlet that will survive all possible abuse by motor vehicles, but he should be certain to include vehicle loads in the design.

4. *Internal Energy Dissipators*

Research conducted in the 1960s at the Virginia Polytechnic Institute and State University (VPI) established that excess energy in storm water flowing down steep drainage channels could be dissipated by constructing roughness elements within the channel.

Initial tests were conducted on partially-full pipes, and subsequent tests were conducted for full-flow conditions occurring near the outlet

end of the pipe at maximum discharge. By allowing the culvert to approach full flow, it was found that velocity reductions could be effected with the roughness elements without increasing pipe diameter (American Concrete Pipe Association 1988).

D. Detention/Retention Dams and Ponds

The design of detention/retention dams and ponds probably requires more geotechnical input than any other aspect of storm drainage design. Geotechnical considerations for design of these facilities include embankment stability, pond-related seepage, foundations of appurtenant structures, sources of embankment materials, and embankment placement specifications.

The stability of the embankment should be evaluated for all reasonable loading conditions. Those conditions include normal static loading, loading during flood flows, rapid drawdown loading (if the pond drains quickly after each filling), and earthquake loading, if applicable for the project site. If accommodating an earthquake loading is very expensive, the engineer and the owner may elect to evaluate the risk of eliminating the load from the design conditions. In typical embankment dam design, the simultaneous occurrence of an earthquake and a flood is not normally considered. For a detention dam that is normally dry, the principal risks associated with earthquake loading are slope failure, cracking of the dam, or damage to appurtenant structures. If such events occur when the pond is dry, they do not pose any downstream threats and repairs can be made before the next flood occurs. In such circumstances, it may be reasonable for the owner to elect not to have a design that can withstand earthquake loading.

Any evaluation should consider seepage through the embankment, under the embankment through the foundation, and into or through the reservoir bottom or abutments. The significance of any of these potential seepage paths depends on the expected duration of pond storage. If the duration of pond storage is long enough for seepage to occur through either the embankment or its foundations, then the design must include provisions to prevent failure of the embankment by internal erosion or "piping" of the embankment or foundation materials. Such a failure could result in a rapid, uncontrolled release of the stored water, which could flood downstream areas. Measures to prevent piping include impervious upstream blankets or impervious core zones to prevent seepage; or internal filters or internal drains and toe drains to safely control and collect the seepage.

Depending on the composition of the materials in the abutments and the reservoir bottom, seepage through those areas can also cause piping failures, which could result in the loss of part or all of the reservoir. For this case the solutions are similar to those for the embankment.

Another potential threat from seepage through the reservoir bottom or abutments, and to a lesser degree through the embankment and its foundations, is the possibility of a temporary rise in the downstream

groundwater levels because of seepage. The time required for seepage from the impoundment to result in a rise in the groundwater depends on the permeability of the materials through which the seepage must flow. If the groundwater rises, basement flooding and similar problems could occur. Potential solutions to these problems include upstream impervious blankets to prevent the seepage from entering pervious zones, seepage cutoff walls, downstream pumping wells to collect the seepage before it reaches areas where it can cause damage, and increased hydraulic capacity of the spillway and outlet works to shorten the duration of impoundment.

Geotechnical considerations for appurtenant structures include stability and settlement/heave. As for the embankment itself, the appurtenant structures should be evaluated for all reasonable loading conditions. For intake structures, retaining walls and other facilities subjected to lateral loads, the evaluation of stability should address both overturning and sliding. For structures such as retaining walls, the earth pressures are affected by drainage behind the structures. The geotechnical engineer must insure that his recommended pressures are consistent with the drainage provisions incorporated into the design.

It is important that appurtenant structures that pass through the embankment, such as pipe outlet works, be designed in such a manner that they do not promote seepage and piping along the structure-soil interface. In recent practice, the use of seepage collars around such structures has been giving way to the specification of more careful compaction near the structure and the use of filter zones near the downstream ends to control and collect any seepage along the structure.

Design studies should include evaluations of whether or not the proposed sources of embankment material are adequate. After the materials have been selected and the availability of sufficient quantities has been established, the placement specifications should be formulated in a manner to insure that the in-place materials will have the properties assumed in the design. The equipment and procedures specified for embankment construction should be suitable for the type of material and should be as consistent as possible with local practice and with locally-available equipment.

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Chapter 15
CONSTRUCTION CONTRACT DOCUMENTS

I. INTRODUCTION

The purpose of contract documents is to portray clearly by words and drawings the nature and extent of work to be performed, the known or anticipated conditions under which the work is to be executed, contractual requirements, the rights and responsibilities of the various parties to the contract (including the engineer), and the basis for payment (ASCE 1982). Contract documents typically consist of bidding requirements, bid forms, contract forms, conditions of the contract, specifications, addenda, and construction drawings (plans). The contract drawings, conditions of the contract, specifications and other components of the contract documents collectively define the work to be undertaken by the contractor. The drawings, conditions, and specifications are complementary; what is called for by one is to be executed as if called for by all (Abbott 1963).

This chapter provides an overview of the elements of an acceptable package of contract documents for stormwater facility construction, and closes with a checklist, which has been adapted from the Construction Specifications Institute Manual of Practice (Wright Water Engineers 1986). Thoughtful and meticulous assembly of contract documents can be of paramount importance in reducing or eliminating the professional liability and insurance problems that confront every public works engineer and private consultant.

II. CONTRACT DRAWINGS

Contract drawings must depict graphically the project's layout and the work to be done to the owner, to project reviewers, to the bidders, and later to the construction observer and the contractor. Drawings normally consist of plan and profile views and cross-sections, and in-

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clude both general and detailed drawings. General drawings provide an overview of the facility being constructed and are drawn at a relatively small scale.

Drawings are usually prepared by the design engineer, and together with the written contract conditions and specifications, form the physical dimensional description for the contractor's estimate and for the construction contract itself. Properly formulated design drawings stand on their own and require the contractor to spend relatively little time searching the technical specifications for a description of a particular task.

A. Drawing Preparation

Contract drawings generally are prepared on a medium that facilitates reproduction. For storm sewers or open channels, printed plan and profile blank sheets are available, or they may be specially printed and titled to reduce drafting time. The plan view should be drawn on the top half of the sheet with the profile provided directly below.

Contract drawings should be prepared carefully in a neat, legible fashion. Hastily produced, sloppy drawings can lead to mistakes during construction. Engineers are advised to develop in-house procedures for checking every aspect of the drawings.

B. Contents

The most logical arrangement for a set of contract drawings proceeds from general views to more specific views.

1. Title Sheet

The title sheet should identify the project with the following minimum information:

(a) Project name.
(b) Contract number.
(c) Federal or state agency project number (if applicable).
(d) Owner's name.
(e) Owner's officials, key people, or dignitaries.
(f) Design engineer's name.
(g) Engineer's project number.
(h) Plan set number (for distribution records).
(i) Professional engineer's seal and signature.

2. Title Blocks

Each sheet should have a title block, which includes:

(a) Sheet title and number.
(b) Project name.
(c) Federal or state agency project number (if applicable).
(d) Owner's name.

(e) Design engineer's name.
(f) Engineer's project number.
(g) Scale (if not provided elsewhere on drawing).
(h) Date.
(i) Designer, drafter, and checker identification.
(j) Revisions block.
(k) Sign-off space for owner's chief engineer or district superintendent, if applicable.

3. *Index/Legend*

Contract drawings should contain an index listing all the drawings in the set by title and drawing number in the order of presentation. This index should normally be located on the drawing following the title sheet, although for small projects the index can be on the title sheet.

A legend should be included to provide a description of all symbols used on the drawings.

4. *Location Map*

A location map should be provided showing the location of all work in relation to surrounding features and major routes of access to the project site. This map should either be on the title sheet or on the index/legend sheet.

5. *Subsoil and Groundwater Information*

The sampling locations, logs, and laboratory analyses of soil borings obtained during the design phase of a project should be included, together with a Design Summary Report giving all of the assumptions used by the designer in making the contract documents.

When possible, drawings and specifications should indicate where special construction is required to accommodate known subsurface conditions. Similar precautions apply to groundwater. With the exceptions of swamp or valley bottom conditions in which submerged excavation is a certainty, a note that groundwater may be encountered during excavation is advisable.

6. *Survey Control Data*

Necessary survey control information should be shown. Baseline bearings and distances should be included with references to permanent physical features. Vertical control points, or bench marks, should be indicated, and the datum plane used for determining these elevations defined. A note indicating the dates of the ground survey and aerial photography should be included. Frequently, there will be inconsistencies among county, city, and U.S. Geological Survey datum planes. When this is the case, the drawings should indicate which datum plane is relevant, and should provide instructions for converting to other datum planes if necessary.

7. *Closed Conduit, Open Channel, and Detention Pond Stormwater Plans and Profiles*

Plan and profile views are normally presented on the same sheet, and stationing of the plan and profile should coincide. A continuous strip map usually indicates the locations of all work in relation to surface topography and existing facilities. The location of underground and overhead utilities that cross or are near the proposed construction route also should be shown, as should any other structure that could affect construction.

Surveyed baseline stationing off the alignment may also be given on the plans, but it should not be substituted for stationing along the center line of the sewer or channel.

Stationing indicated on construction drawings for location of manholes and wye-branches or house connections should be considered approximate only and so noted. Locations of junction structures must be given on construction drawings. Match lines should be used and should be easily identifiable. Special construction requirements, such as sheeting to be left in place, should be shown on the drawings. Where interference with other structures exists, explanatory cross-sections and notes should be included. Such cross-sections, often enlarged in scale, should be identified as to specific location and, if possible, should be placed on the plan/profile drawing near the relevant section.

For storm sewers, the profile is a convenient place to show pipe size, length and slope, the strength or type of pipe, the locations of special structures and appurtenances, and crossings of utilities and other drainage pipes. Drawings for open channels provide such characteristics as bottom and top width, side slopes and depth, on a reach by reach basis, or should refer the reader to cross-sections elsewhere in the drawings.

8. *Profile and Section Views*

When common pipe sizes and materials are specified, no sewer cross-sections need be shown. For cast-in-place concrete sections, however, complete dimensions and all reinforcing steel should be shown in the drawings (ASCE 1982).

Sectional views of open channels, detention ponds, or related dams or embankments should always be provided, and at multiple locations if necessary. Of particular importance for open channels are the transition sections. There should be at least one cross-section for each outlet or control section.

9. *Details*

Separate sheets depicting details normally follow the plan/profile sheets.

10. Special Details

Details not covered on standard detail sheets should be given on sheets entitled "Special Details." As an example, for sewer projects the following would be included:

(a) Special structures—full details so that the finished work can be structurally sound and hydraulically correct.
(b) Special castings—sufficient details for the manufacturers to prepare shop drawings. Standard casting items, such as manhole frames, covers, and steps, can be identified by reference to a manufacturer's catalog number in the specifications.
(c) Restoration—complete details for pavement, sidewalk, and curb repairs.

11. Record Drawings

During project construction, the contractor should measure and record the "as-installed" locations of all appurtenances being buried that may have to be located in the future. All changes from the original plans, and locations of unexpected rock, seepage, or other unanticipated natural problems, should also be recorded. Extreme deviations in quantity or character of work can entitle the contractor to scope and payment modifications, so it is very important for the inspector to keep accurate logs and records to document the job fully in the event of a dispute.

Contract drawings should be revised to reflect field information after the project is completed and a notation such as "Record Drawing—based on approximate field observations" should be made on each sheet. Sets of such revised drawings should become a part of the owner's and the engineer's permanent records.

III. SPECIFICATIONS

The sample bidding forms, agreement forms, conditions of the contract, and descriptions of expected construction details and minimum criteria for technical performance or material to be provided, is referred to collectively as the Specifications.

Engineers preparing the specifications should consult the format recommended by the Construction Specifications Institute (CSI undated) as a checklist to assure that all components are provided. Although some flexibility on the part of the engineer is acceptable, failure to adhere to a reasonably consistent format can lead to specifications that are confusing or otherwise inadequate. The components of the specifications should be standard in nature and readily understood by all potential bidders. The major components of the specifications are:

(a) Addenda.
(b) Bidding requirements.

(c) Contract forms.
(d) Conditions of the contract.
(e) Detailed specifications.

These documents should be assembled in this order and should be prefaced with a cover page, title page, and table of contents. It is desirable to have all components of the specifications bound in a single volume, though for large projects, that may not be possible. In such cases a "standard specification" should be developed and bound separately. It should then be incorporated by reference in multiple-volume specifications, each pertaining to a single component of the project.

Documents contained in the specifications set forth the details of the contractual agreement between the contractor and the owner. The documents describe the work to be done (complementing the information provided on the drawings), establish the method of payment, set forth the details for performance of the work including necessary time schedules and requirements for insurance, permits, and licenses, and delineate responsibilities of the various parties involved with the project.

A. Addenda

After the initial set of plans and specifications have been issued to bidders, the necessity to modify contract documents frequently arises. Changes may be made to:

(a) Correct errors and omissions.
(b) Clarify questions raised by bidders.
(c) Issue additions and deletions.
(d) Describe changes in design based on new geotechnical, hydrologic, or other information.
(e) Comply with revisions in the requirements of the owner.

Procedures for issuing addenda are described in the instructions to bidders, and a space for the bidder to acknowledge receipt of addenda is provided on the bid form. Addenda must be issued sufficiently in advance of the bid opening to give bidders time to use the information in bid preparation. An addendum issued after contract award usually results in a change of the contract price and requires the issuance of a contract "change order." The engineer must always obtain the owner's approval before authorizing an addendum.

B. Bidding Requirements

The specifications should contain all bidding requirements, including the invitation to bid, instructions to bidders, and a bid form.

1. Invitation to Bid

The purpose of the invitation to bid (or advertisement) is to inform prospective bidders that a contract is to be awarded and that bids are

being solicited by the owner. The advertisement should be included in local, regional, or national newspapers (depending on the size of the project), as well as in periodicals or other journals with wide circulation to potential bidders. It may be mailed to contractors with proven capability to conduct the work, and may also be posted in a public place, such as a post office or municipal building. The advertisement should be published sufficiently in advance of the bid opening to allow time to prepare estimates, obtain prices and sub-bids for specialty work, and to make other arrangements necessary to arrive at the bid amounts. One month is generally considered the minimum time acceptable between issuance of an advertisement and bid opening.

The advertisement should be brief and clearly written. Essential elements include:

(a) Brief description of the work and its location. This should be written to attract the attention of only those interested and qualified to bid.
(b) Name and address of the owner.
(c) Name and address of person authorized to receive bids.
(d) A clear description of any statutory requirements regarding preference to local contractors, labor and materials, and/or set-aside programs.
(e) The place, date, and hour of the opening of bids.
(f) Principal items of the work with approximate quantities involved. This informs the contractor immediately whether his equipment, organization, and experience are suitable for the work.
(g) Bid deposit. The amount of the bid deposit and whether it is to be cash, a certified check, or a bid bond should be stated, as well as the provisions for the return of the bid deposit to unsuccessful bidders.
(h) Information relative to the plans and specifications. It should be stated where the plans and specifications may be obtained or examined. This will usually be in the offices of the owner or the engineer, or sometimes in the offices of the contractor or trade associations. Charges or deposits required for the plans and specifications should be noted, together with provisions for refund when the documents are returned.
(i) Name of the engineer and the owner or their authorized representatives. In the latter case, the authorization should be stated.

2. *Instructions to Bidders*

The instructions to bidders, also known as information for bidders, is a document that furnishes information on the unique features of the work and detailed instructions on the procedure to be followed in submitting bids. This is desirable to assure that all bidders receive uniform treatment and to provide a common basis for bid preparation. The information given is similar in character to that in the advertisement, but is more explicit and in greater detail.

Instructions to bidders should do the following:

(a) Summarize the major components of the specifications.
(b) Describe technical aspects of the bidding process. For example, who from the bidding company is required to sign the proposal?

(c) State whether the bids are to be on a lump-sum or unit-price basis, and whether they are for the entire project or certain parts only. Note also should be made of any alternate bids requested.
(d) Stipulate requirements for an accompanying bid bond, if any, including the nature and amount of the bond.
(e) Describe the amount and type of performance and labor and material payment bonds required.
(f) Describe procedures to be followed if alternate or substitute materials or processes from those described in the specifications are to be proposed.
(g) Describe any required elements of a statement of competency, including such items as descriptions of work performed in the previous five years, descriptions of equipment available to use on the project, or recent financial statements.
(h) Request or require documentation of contractor's familiarization with the work in question, including not only the physical work to be performed but also the applicable federal, state and municipal laws, regulations and ordinances pertaining to labor, materials, specifications, and contract matters that may affect the proposed work.
(i) Inform the contractor that the owner reserves the right to revise or amend any one of the stated parts of the contract documents prior to the date set for opening the proposals.
(j) Inform the bidder of the time for completion of the work and of the method of payment.
(k) Announce the time and place for proposal submission, the packaging required for the proposal, and any other unique bidding factors.
(l) Outline procedures for bid submission or bidder changes or withdrawals.
(m) Describe procedures to be followed for acceptance of a proposal by the owner, and the owner's reservation of rights to reject any or all proposals, to waive inconsistencies and informalities, and to award the contract on the basis of the owner's determination of the lowest responsible bidder (i.e., the one that best serves the interests of the owner).
(n) Inform bidders of the requirements regarding start-up time, insurance, and other factors. These should be described in the final section of the instruction to bidders.

3. *Bid Form*

The purpose of providing a bid form is to insure that all bidders submit prices on a uniform basis, so as to facilitate comparison. The bid form is advantageous to both the owner and the bidder, because the form tends to insure accuracy and prevent omission. The bid form should contain the following elements:

(a) Price for which the contractor offers to perform the specified work.
(b) Time of completion.
(c) Bid Deposit.
(d) Agreement by contractor to post required performance and labor and material payment bonds upon award of the contract.
(e) List of addenda to the plans and specifications that were considered when the bid was prepared.

(f) List of subcontractors.
(g) Experience record, financial statement, and plant and equipment questionnaire, when required.
(h) Declaration that no fraud or collusion exists, with particular reference to illegal relationships between the bidder and representatives of the owner, pooling of bids by several bidders, straw-man bids submitted by an employee or other representative of the bidder, and similar illegal acts.
(i) Statement that the site has been examined and that the plans and specifications are understood by the bidder.
(j) Signature and witnesses.

4. *Responsibility for Accuracy of Bidding Information*

On some types of stormwater-related construction work, the amounts of bids will depend on local conditions at the site, some of which cannot be precisely determined in advance. For example, it may be impossible to obtain a sufficient number of soil borings to adequately assess the existing subsurface conditions. Available geotechnical information may not reveal all conditions and may be misleading or subject to incorrect interpretation. Similarly, unexpected high groundwater table conditions can increase the difficulty of a job.

The construction of hydraulic engineering projects is frequently a risky undertaking, and the three parties involved in the planning, design, and construction of hydraulic works (the owner, the designer, and the contractor), each have certain responsibilities they must assume.

The owner desires a functional project designed and constructed in accordance with generally-accepted industry standards. The owner should be willing to pay for the work, including any overruns due to differing site conditions. He should not expect the designer or the contractor to finance the project when cost overruns were not caused by their errors or omissions.

The designer has a responsibility to provide a cost-effective design and to disclose all of the assumptions used in preparing that design. He should be responsible for any errors or omissions made in the course of the design.

The contractor has the responsibility to make a fair bid on the project and to perform the work in a timely manner. He should be responsible for any costs due to his negligence.

These simple responsibilities and relationships between the three parties can deteriorate to the point where the parties become adversaries. These problems may be alleviated by incorporating a "Design Summary Report" (which gives all project design assumptions) into the contract documents and by including three significant features in the contract documents. These are a Pre-qualification of Bidders, Escrow Bid Documentation, and the Disputes Review Board.

The Pre-qualification of Bidders and their subcontractors is used to demonstrate whether or not the contractor and his staff have previous

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experience on similar projects. Those who do not show the requisite experience should be eliminated from the bidding process.

The Escrow Bid Documentation is used to determine equitable price adjustments for extra work and changed conditions. This documentation consists of a detailed breakdown of how the Contractor prepared his bid including all assumptions used in preparing his estimate. This documentation is then held in escrow for use in settling any disputes during the course of the work.

The Disputes Review Board is a board comprised of three highly qualified members, none of whom are present or former employees of the concerned parties, one appointed by the owner, one appointed by the contractor, and a third appointed by the owner and the contractor. The third member serves as chairman. The Board meets regularly at the construction site and reviews and makes recommendations regarding disputes between the owner and the contractor.

A sample specification for use in the Pre-qualification of Bidders, Escrow Bid Documentation, and developing a Disputes Review Board is included in Westfall (1987).

The engineer's estimate of quantities is given solely for the purposes of indicating the scope of work and for comparing bids for unit price contracts. The unit prices in the bid are binding on the contractor until the variation in estimated quantities threshold is met. It also has been held that the engineer's estimate is a representation, which, if grossly in error, may provide sufficient grounds for the contractor to obtain a change in unit prices.

Without unusually detailed engineering analysis, it is impossible to quantify construction quantities precisely for large stormwater detention ponds, channel widening projects, or even storm sewer projects. Nonetheless, the goal of the engineer's estimates is to provide a reasonable assessment of the probable magnitude of the project. If quantity requirements in the field exceed the engineer's estimates by about 25% (this percentage is subject to variation), it may be necessary to readjust the contract unit prices. The contract documents should provide for such readjustment.

C. Contract Forms

The basic essentials of a valid contract consist of:

(a) The conditions precedent to the agreement.
(b) A statement of the work to be done.
(c) The time in which it is to be done.
(d) The compensation to be paid for its performance.
(e) Signatures of the contracting parties affirming their agreement as to the conditions imposed by the contract.

These items, together with the plans and specifications, will rarely produce a contract sufficient in scope for complex stormwater projects. It will be necessary for the contract to cover such items as liquidated damages, bonus clauses, escalation clauses, changes or extra work, and

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other factors. No major construction contract should be entered into until counsel for both the owner and the contractor are satisfied with all contract provisions.

1. *Form of Contract*

Contracts for construction work follow legal practice and assume many different forms, determined principally by the laws of the local jurisdiction and state in which the contract is executed. The contract must cover all items contained within the specifications.

There are two basic components of the contract. The first contains the basic articles, such as the scope of work, compensation, and completion time, which together are referred to as the "Agreement." The second part of the contract is referred to as the "Conditions" of the contract, which are discussed later.

2. *Surety Bonds*

Contractors are generally required to furnish surety bonds as a guarantee of faithful performance of the contract and payment of bills for labor and materials. Surety bonds are usually issued by a bonding company, although private individuals may serve as sureties or the contractor may furnish his own surety by depositing acceptable property or collateral with the owner.

There are three parties to the execution of a bond:

(a) The principal (contractor) on whose behalf the bond is written and whose performance is guaranteed.
(b) The obligee (owner and persons furnishing labor and materials on the project) in whose favor the bond is written.
(c) The surety (bonding company) who acts as guarantor for the principal and who is obligated to make good to the obligee any default on the part of the principal.

The three most common types of surety bonds are bid bonds, performance bonds, and payment bonds. The bid bond is submitted with the proposal and basically guarantees that the bidder will enter into the contract if his proposal is accepted. If the contractor has signed the contract and has received authorization to commence work from the owner, and then reneges on his obligation and does not start work, the owner is paid by the bonding company an amount equivalent to the bid bond. The performance bond guarantees that the contractor will perform the contract in accordance with the stipulations of the contract. The performance bond is usually established for the full amount of the contract. The payment bond assures that the contractor will pay all expenses he incurs as the contract is performed, thereby rendering the owner harmless for claims and liens that would arise if the contractor defaulted on his financial obligations. The payment bond should be at least 50% of the value of the contract price.

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3. *Special Forms*

Unusual aspects may be associated with the contract and may warrant the use of special contract forms. Such forms are referred to as "special forms" and should be attached to the general contract. Special forms will often be associated with federal, state or local government work.

D. Conditions of the Contract

The conditions of the contract fulfil many important functions, including the following:

(a) Provide definitions of important words, terms, and phrases.
(b) Provide an outline of the contract documents.
(c) Establish the rights and responsibilities of the owner, the contractor, subcontractors, and other salient parties.
(d) Provide instructions on how to implement provisions of the contract.
(e) Discuss such aspects as work supervision, changes in work, claims for extra cost, delays and extensions of time, owner's right to terminate contract, insurance bonds, damages, and other matters.

Although the conditions of the contract often constitute the bulk of the contract documents, they should be administrative in nature and should not include detailed specifications for materials or workmanship. These should be provided in the technical specifications.

Construction contracts generally include both general conditions and supplementary (special) conditions. General conditions cover aspects of job management unrelated to the specific project, but which apply to all construction projects, such as insurance requirements, rights and responsibilities of the contractor and of the owner, or bond requirements. Supplementary conditions relate to the particular project, including such things as special environmental considerations associated with the project, provisions for obtaining water at the site, the role of governmental entities, and other such factors.

E. Detailed Specifications

Written instructions that accompany the project drawings are referred to as "Detailed Specifications." The drawings and detailed specifications provide a complete summary of the technical requirements of the work to be performed. Because most contracts stipulate that, in the cases of conflict between specifications and drawings, the provisions of the specifications will govern, it is of great importance that the engineer develop specifications that are clear, concise, and comprehensive.

There are two categories of detailed specifications: (1) general provisions, which apply to the work as a whole and cover such things as a summary of the work to be performed, measurement of payment, quality control, and related subjects; and (2) technical provisions, which describe technical details for construction processes such as earthwork, boring and jacking, or tunnelling, and for materials such as steel, concrete, metals, or grouted riprap.

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The Construction Specifications Institute lists nine items that should be included in the general provisions of the specifications (CSI undated):

(a) Summary of work.
(b) Alternatives.
(c) Measurement of payment.
(d) Project meetings.
(e) Submittals.
(f) Quality control.
(g) Temporary facilities and controls.
(h) Materials and equipment.
(i) Project close-out.

A tenth item, safety, should also be included, with proper references to appropriate federal or state OSHA requirements.

Other items can include electrical service, overhead structures, surveys, lines and grades, access to site, pre-construction conference, time for completion of work, inspection requirements, protection of public and private property, water for construction, erosion control, wildlife mitigation efforts, or dust control.

The technical provisions contain the detailed instructions necessary to obtain the desired quality and service in the finished product. In addition to providing detailed instructions, these portions of the specifications also provide for inspection and testing during construction to assure that the project is constructed in accordance with the contract documents. The technical provisions of the specifications must be developed in close harmony with the drawings to avoid possible conflicts.

Materials are commonly specified by reference to the specifications of the American Society for Testing and Materials (ASTM), American National Standards Institute (ANSI), American Concrete Institute, American Water Works Association, or other similar organizations. Although frequent references are made in various sections of the technical provisions to standard specifications, it is recommended that the complete title, serial number, and date of issuance or revision for each standard specifications be cited in full in the general provisions.

The Construction Specifications Institute provides twenty-one categories of site-work specifications, as well as specifications for concrete, metals, and finishes. These are provided in the checklist at the end of this chapter.

There are three broad categories of technical specifications:

(a) Specifications for materials and workmanship that place responsibility on the contractor for furnishing materials and workmanship that will result in a structure of suitable character.
(b) Specifications for the overall performance of the finished product, when the desired operating characteristics of the facility can be measured by specific tests; these specifications are frequently used for machinery such as pumps and motors.
(c) Specifications used for construction work based on the selection of proprietary products in the open market. No control can be exerted

over the manufacturer of proprietary articles, and the specifications may merely identify a desired item known to be satisfactory for the purpose intended. Quality and performance tests or standards may also be prescribed.

The most important principle in specification writing is: within the bounds of reasonableness, only the desired results should be specified, and the contractor should be allowed maximum flexibility to obtain the desired results. Standards of workmanship should be described in specific terms when feasible, but specification of construction methods and safety procedures should always be avoided.

IV. CHECKLIST

The following checklist can be used as a guide for determining the completeness of construction documents for stormwater projects:

A. Bidding Requirements

1. *Invitation to Bid*

(a) Identification of owner or contracting agency.
(b) Name of project, contract number, or other positive means of identification.
(c) Time and place for receipt and opening of bids.
(d) Brief description of work to be performed.
(e) When and where contract documents may be examined.
(f) When and where contract documents may be obtained, and the deposits and refunds therefor.
(g) Amount and character of any required bid deposit.
(h) Reference to further instructions and legal requirements contained in the related documents.
(i) Statement of owner's right to reject any or all bids.
(j) Contractor's registration requirements.
(k) Bidder's pre-qualifications, if required.
(l) Reference to special federal or state aid financing requirements.

2. *Instructions to Bidders*

(a) Instructions regarding bid form to include, at a minimum, the format of preparation, signature(s) required, time and place for submittal; instructions on alternatives or options; and data and formal documents to accompany bids.
(b) Bid security requirements and conditions regarding return, retention, and forfeiture.
(c) Requirements for bidders to examine the documents and the site of the work.
(d) Required use of stated quantities in unit price contracts.
(e) Withdrawals or modifications of bid after submittal.

(f) Rejection of bids and disqualification of bidders.
(g) Evaluation of bids.
(h) Award and execution of contract.
(i) Actions to be taken in case of failure of bidder to execute contract.
(j) Instructions pertaining to subcontractors.
(k) Instructions relative to resolution of ambiguities and discrepancies during the bid period.
(l) Contract bonding requirements.
(m) Governing laws and regulations.

3. *Bid Form*

(a) Identification of contract.
(b) Acknowledgment of receipt of addenda.
(c) Bid prices (lump sum or unit prices).
(d) Construction time or completion date.
(e) Amount of liquidated damages.
(f) Financial statement.
(g) Experience and equipment statements.
(h) Subcontractor listing.
(i) Contractor's statement of ownership.
(j) Contractor's signature and seal.
(k) Non-collusion affidavit.
(l) Consent of surety.

B. Contract Forms

1. *Form of Agreement*

(a) Identification of principal parties.
(b) Date of execution.
(c) Project description and identification.
(d) Contract amount with reference to the contractor's bid.
(e) Contract time.
(f) Liquidated damage clause, if any.
(g) Progress payment provisions.
(h) List of documents comprising the contract.
(i) Authentication with signatures and seals.

2. *Bonds*

(a) Performance bond.
(b) Labor and material payment bonds.
(c) Maintenance and guarantee bonds (if required).

3. *Special Forms*

C. Conditions of Contract

1. *General Conditions*

2. *Supplementary Conditions*

Perhaps combined with General Requirements of the Specifications to avoid redundancy.

D. Detailed Specifications

1. *General Provisions*

(a) Summary of work.
(b) Alternatives.
(c) Measurement of completed work and payment (many officials include this in the particular work item).
(d) Project meetings.
(e) Submittals.
(f) Quality control.
(g) Temporary facilities and controls (protection).
(h) Material and equipment.
(i) Project closeout.

2. *Technical Provisions—Site Work and Materials (see also ASCE 1969)*

(a) Existing utilities and underground structures.
  (1) Protection.
  (2) Relocation.
(b) Clearing.
  (1) Tree removal.
  (2) Pavement removal.
(c) Earthwork.
  (1) Excavating, backfill, and compacting.
  (2) Limits on trench width.
  (3) Spoil placement.
  (4) Preparation of trench bottom.
  (5) Pipe bedding.
(d) Pipe boring and jacking.
(e) Tunnelling.
  (1) Excavating.
  (2) Casing installation.
(f) Sheeting and shoring.
(g) Rock excavation.
  (1) Definition of rock.
  (2) Excavation.
  (3) Blasting limitations and controls.
(h) Site drainage.
(i) Paving and surfacing.
  (1) Streets and roadways.
  (2) Sidewalks.
(j) Highways and railroad crossings.
(k) Piping materials and jointing.
(l) Manholes and appurtenances.

(m) Pipe laying.
  (1) Control of alignment.
  (2) Control of grade.
(n) Service connections.
(o) Connections to existing sewers.
(p) Connections between different pipe materials.
(q) Concrete encasement or cradles.
(r) Sewer paralleling water main.
(s) Sewer crossing water main.
(t) Repair of damaged utility services.
(u) Acceptance tests.
  (1) Infiltration.
  (2) Exfiltration.
  (3) Smoke.
  (4) Air.
(v) Concrete (CSI Division 3).
  (1) Forms.
  (2) Concrete reinforcement.
  (3) Cast-in-place concrete.
  (4) Concrete curing.
(w) Metal Fabrications (CSI Division 5).
(x) Finishes (CSI Division 9).
  (1) Painting.
  (2) Waterproofing.

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Chapter 16

CONSTRUCTION METHODS

I. INTRODUCTION

The design and construction of drainage systems are so interdependent that knowledge of one is essential to the competent performance of the other. The intent of this chapter is to introduce the engineer to some common construction techniques and to encourage their consideration in the design and development of the construction documents. Local conditions and proper concern for applicable regulations may dictate variations, and the ingenuity of the owner, engineer, and contractor must be accommodated and encouraged if construction costs are to be minimized and a quality job is to result.

Commencement of the construction phase normally introduces a new party, the contractor, to the project. At this stage of the project the division of responsibility and liability must be understood by all. The role of the engineer will normally change from active direction and performance of the design to that of professional and technical observation during construction.

The engineer's representative on the construction site should not be expected to duplicate the detailed inspection of material and workmanship properly delegated to the manufacturer, supplier, and contractor. In fact, the engineer should not agree to monitor, direct, or in any manner have control over the contractor's work, the means, methods, techniques, sequences, or procedures of construction. The engineer's role should be one of identifying and correcting problems as a result of misunderstanding or misinterpreting the contract documents.

Preconstruction conferences are helpful in deciding whether the contractor's proposed operations are compatible with contract requirements and whether they will result in finished construction acceptable to the owner. These joint meetings of the owner, engineer, and contractor should result in definite construction schedules and administrative procedures to be followed throughout the duration of the construction

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Careful attention must be paid to the quality of facility construction. This parking lot was supposed to drain into the adjoining field. Instead, the field drains into the parking lot.

contract. On larger jobs, the geotechnical engineer familiar with the project design should be available to answer questions regarding any special construction techniques. The meeting agenda should include items such as progress schedules, progress payment format and details, method of making submittals for review, and channels of communication. All of these aspects of construction should be settled before construction begins. Where special permits have been issued for construction, specifically in the areas of wetlands or other environmentally sensitive issues, copies of such permits and limiting conditions should be made available to all parties at the meeting.

II. CONSTRUCTION SURVEYS

A. General

Baselines and benchmarks for storm drainage system alignment and grade control should be established along the route of the proposed construction by the engineer, or by the contractor if the work is reviewed by the engineer. All control points should be referenced adequately to permanent objects located outside normal construction limits.

Control points should be identified with both a name and description and should provide both vertical and horizontal control with elevations and coordinates.

B. Right of Way

Acquisition of easement or fee title to rights-of-way to provide adequate working space for construction projects should be completed and rights-of-way cleared before project construction begins. Access, materials storage, and equipment movement space needs should be anticipated.

Project construction in urban areas often requires removal and replacement of fences, landscaping, and even buildings. To avoid construction delays and adverse public reaction, such actions should be pre-arranged with adjacent property owners.

Rights-of-way needed for maintenance may differ from those required for construction. For instance, maintenance vehicle access points may be needed, and should be designed for minimum interference with drainage facility performance.

C. Preliminary Layouts

Prior to the start of any work, work areas, clearing limits, and pavement cuts should be laid out to give proper recognition to, and protection for, adjacent properties. Limits of temporary and permanent easements should be carefully delineated in the field so their relation to permanent improvements can be verified. Access roads, detours, bypasses, and protective fences or barricades also should be laid out and constructed as required in advance of construction. All layout work, if done by the contractor, should be reviewed by the engineer before any demolition or construction begins.

D. Setting Line and Grade

The transfer of line and grade from control points to the construction work is normally the responsibility of the contractor, with spot checks by the engineer as work progresses. The preservation of stakes or other line and grade references provided by the engineer is similarly the responsibility of the contractor. In most cases, there is a charge for reestablishing stakes carelessly destroyed by the contractor, and the charge is stated as part of the contract agreement.

In general, the line and grade for the sewer may be set by one or a combination of the following methods:

(a) Stakes, spikes, or crosses set on the surface at an offset from the sewer centerline.
(b) Stakes set in the trench bottom along the sewer line as the rough grade for the sewer is completed.
(c) Elevations given for the finished trench grade and sewer invert while sewer laying progresses.

(d) Laser beam.
(e) Stakes set on the surface at an offset from the top of slope indicating cut or fill and slopes.

Method (a) generally is used for small diameter sewers. Methods (b) and (c) are used for large sewers or where sloped trench walls result in top-of-trench widths too great for practical use of short offsets or batter boards. Method (d) is independent of the size of sewer. Method (e) is used for open channel conveyance systems that have sloped banks or deep excavations where side slopes are specified prior to construction.

In method (a) stakes, spikes, or crosses are set on the opposite side of the trench from which excavated materials are to be placed and at a uniform offset, insofar as practicable, from the sewer centerline.

The line and grade may be transferred to the bottom of the sewer trench by the use of batter boards, tape and level, or patented bar tape and plumb bob unit.

Batter boards and batter board supports must be suspended firmly across the trench and be adequate to span the excavation without measurable deflection. If the spanning member is to be the batter board, it is set level at an even foot (or other convenient unit of measurement) above the sewer. Preferably, the spanning member is used as a support only and a 1 inch batter board is nailed to it with one edge in a true vertical plane at the centerline of the storm sewer. A nail then is driven in the vertical edge of the batter board at an even foot above sewer grade. A string line is drawn taut across at least three batter boards.

The sewer centerline is then transferred to the trench bottom with a heavy plumb bob held lightly against the string line. Grade is transferred to the sewer invert with a grade rod equipped with a suitable metal foot to extend into the end of the pipe. For steep grades, it is advisable to fasten a bullseye level to the grade rod to assure that the rod is held plumb. For ease in reading, the grade rod may be marked at subgrade, finish grade, and invert grade. The line and grade of the string line should be checked by observation for possible error in cuts or in establishing the batter boards. Periodic inspection should be made during sewer laying to insure that the set line and grade have not been disturbed.

Another method of setting grade is from offset crosses or stakes, or from offset batter boards and double string lines and the use of a grade rod with a target near the top. When the sewer invert is on grade, a sighting between grade rod and two or more consecutive offset bars or the double string line will show correct alignment.

The transfer of surface references to stakes along the trench bottom is in some instances permitted, but the use of batter boards is preferred. If stakes are established along the trench bottom, a string line should be drawn between not less than three points and checked in the manner used for batter boards.

When trench walls are not sheeted but sloped to prevent caving, line-and-grade stakes are set in the trench bottom as the excavation

proceeds. This procedure requires a field party to be at the job site almost constantly.

Another method, applicable to large diameter sewers or monolithic sections of sewers on flat grades, requires the line and grade for each pipe length or form sections to be set by means of a transit and level from either on top or inside of the completed conduit.

In the construction of large sewer sections in an open trench, both line and grade may be set at or near the trench bottom. Line points and benchmarks may be established on cross bracing where such bracing is in place and rigidly set. Later, alignment and grade must be determined by checking the setting of the forms.

A method quite widely used is laser beam control. A laser is a device that projects a narrow beam of light down the centerline of the sewer pipe. It is usually set up in the invert of a manhole and then aligned horizontally. The proper slope is established by adjusting a dial on the machine and aiming the laser. A check elevation should be set about 100 feet from the manhole to assure that the proper slope is being maintained by the beam of light. A target set in the pipe centerline is then used to align the end of each pipe section. Care should be exercised in the use of the laser since temperature affects the aiming of the unit.

Where tunnel construction is an extension of a sewer of sufficient size without change of alignment, the initial line and grade for the tunnel work may be established by extending lines and grades through and forward from the completed portion.

When tunnelling begins from an isolated shaft, great care must be taken in transferring line and grade from the surface. If tunnelling from any one shaft extends more than several hundred feet from the shaft, and especially if the alignment is curvilinear, it may be desirable to verify the vertical and horizontal alignment after each two or three advances. Normal deviations in vertical and horizontal alignment are anticipated and can be adjusted for in the final tunnel lining process.

Slope stakes are generally used to provide horizontal and vertical information to equipment operators on large scale open excavations. These are frequently open channel sections but may be large sewer pipes or box culverts requiring large scale excavations due to their size or depth. The stakes are located at an offset outside the top of the intersection of the excavation with the existing ground surface. Stakes are marked with centerline station, the offset, the required cut or fill to the excavation bottom and a slope at which the bank is to be constructed. The tops of both banks are slope staked in this fashion at regular intervals along the proposed centerline.

When the excavation is sufficiently large, it also may be possible to stake the sewer centerline and the toe of slope. This will provide the operator with additional control with which to perform the construction. These stakes will be destroyed prior to finish grading, but will provide enough information for rough grading. These stakes are marked with centerline station, their location on the bottom (i.e. centerline, toe of slope, etc.) and the appropriate cut or fill.

III. SAFETY

A. General

Safety on the construction site is a critical concern of all participants in the project. The responsibility for safety on the project site generally lies with the contractor who has direct control of the construction activities. However, all who are on the site have an interest in maintaining a safe working environment. A safety memo or manual setting forth procedures to be followed during an emergency should be present at all construction sites.

Many organizations such as the Federal OSHA and similar State organizations have developed standards specifying minimum provisions for site safety. Of greatest interest on sewer projects are those provisions dealing with excavating, trench supports, and tunnelling and blasting. Hard hats and other personal safety equipment should be required.

B. Excavation

The most critical safety issue for excavations is the stability of open cut slopes. In no case should an unsupported slope be excavated at an angle steeper than the material's angle of repose. If right-of-way or other constraints prohibit the flattening of slopes, shoring and bracing or trench boxes (where allowed) should be provided to allow for the maintenance of safe working conditions.

Precautions should also be taken to prevent unwarranted access to excavations. If necessary, fences, barriers or guardrails should be installed to protect unwary passersby.

C. Tunnelling

In addition to normal safety-related issues associated with movement of materials and equipment, precautions must be taken to insure that adequate ventilation and roof and wall support are provided in tunnelling operations.

Ventilation takes on additional importance when underground operations generate large volumes of dust or during blasting operations. The buildup of flammable, explosive, or toxic gasses such as methane, natural gas, or sulphur dioxide must be prevented. The venting and disposal of tainted air from the tunnel must meet local air and safety standards.

D. Blasting

"Pre-blast" surveys are mandatory wherever the construction adjoins existing residences or other developed properties. Permission should be obtained, if possible, to enter homes for videotaping and/or photographs. The likelihood of claims in this area is great.

The transport, handling, and storage of explosives must be carefully directed and supervised to maintain safe conditions. The handling of blasting caps is also critical and must be performed by personnel certified under local ordinances.

IV. SITE PREPARATION

The amount of site preparation required may be quite variable. In some cases a significant portion of project cost may be expended on site preparation.

Several owners, engineers, and contractors have adopted a practice of assembling extensive photographic or videotape evidence of preconstruction condition of sidewalks, driveways, street surfaces, building facades, etc., to minimize post-construction claims by residents and others for construction-related damages. These evaluations are performed prior to the commencement of any construction activities and might be classified as pre-construction reconnaissance.

Operations that may properly be classified as site preparation are clearing and grubbing; construction of access roads, detours, and bypasses; control of existing drainage; location, protection, or relocation of existing utilities; and pavement cutting. The extent and diversity of these operations make extensive additional discussion thereof impractical here. Note, however, that the success of the contractor in keeping the project on schedule depends to a great degree on the thoroughness of the planning and execution of the site preparation work.

A. Clearing and Grubbing

Clearing and grubbing operations are intended to remove unwanted vegetation and unusable surface materials from the site prior to the commencement of other construction operations. The work will generally consist of the removal of all surface objects (debris), trees, stumps, brush, vines, and other objects not designated to remain. It will often include the demolition of existing structures within the storm sewer alignment to facilitate construction. The contractor should be responsible for the proper disposal of materials from the clearing and grubbing operations unless the contract documents indicate reusing some of the cleared materials.

B. Access

Access to the site should be coordinated between the contractor, the engineer, and the owner to minimize construction problems and impacts on adjacent property owners and public streets. On large projects and on some complex smaller projects, the contractor should be encouraged to analyze the access to various sites within the construction limits to anticipate and resolve problems with material delivery and equipment movement.

C. Utilities

In all excavations, extreme care should be taken to properly locate, support and protect existing utilities, particularly those underground. The owners of the utilities should be contacted before the start of excavation. During design, the utilities should be field-located by staking for horizontal location and excavation where necessary for vertical location. Potential conflicts with existing utilities should be addressed during the design of the project. The contractor should be required to notify all utility owners and to field-verify the location of all utilities prior to the commencement of construction (many utilities will provide an on-site representative to work with and assist the contractor). Any deviations from the lines and grades shown on the drawings should be brought to the immediate attention of the engineer.

Project designs should identify and provide specific construction procedures for utility crossing protection. Often overlooked are "unusual" utilities such as petroleum or gas pipelines, underground electrical distribution, underground cables belonging to private or commercial communications systems or telemetry. Larger conflicting utility installations such as telephone junction boxes, electric utility transformer stations, high pressure gas transmission lines, etc, should be identified and relocated by their owners or under separate contract. If possible, relocations should be completed prior to sewer construction activities to minimize delays or disruption of the sewer construction.

It may even be infeasible to adjust or relocate some existing utilities. Such situations must be identified during project design. There is no excuse for the discovery of such situations during project construction.

Even with these precautions, damage or interruption is a possibility, and drainage system managers should have contingency measures planned and ready for execution.

V. EXCAVATION

A. General

With favorable ground conditions excavation may be accomplished in a single operation; under more adverse conditions it may require several steps. In these circumstances, excavation operations can include stripping, drilling and blasting, and trenching and shoring. In unusual circumstances or complex excavations, hauling and stockpiling of excavated materials may be a necessary part of the excavation process. Excavation work should not be commenced until completion of the site preparation operations.

B. Stripping

Stripping may be advantageous or required as a first step in excavation for a variety of reasons, the most common of which are:

(a) To remove topsoil or other materials to be saved and used for site restoration;
(b) To remove material unsatisfactory for backfill to insure its separation from usable excavated soils;
(c) To remove material having a low bearing value to a depth where there is material capable of supporting heavy construction equipment;
(d) To make it easier to charge drill holes.

In the case of topsoil removal, provisions for the stockpiling of the usable topsoil must be made. This material should be free of large roots or stumps and should be suitable for redistribution subsequent to final grading operations. Stockpiles should be located outside the limits of excavation and as near as possible to the point of final placement.

Unsatisfactory materials should be disposed of in approved on-site locations, as directed by the engineer or owner, or off-site at a location of the contractor's choosing in accordance with applicable local regulations. The off-site location should be reviewed by the engineer.

C. Large Scale Excavation

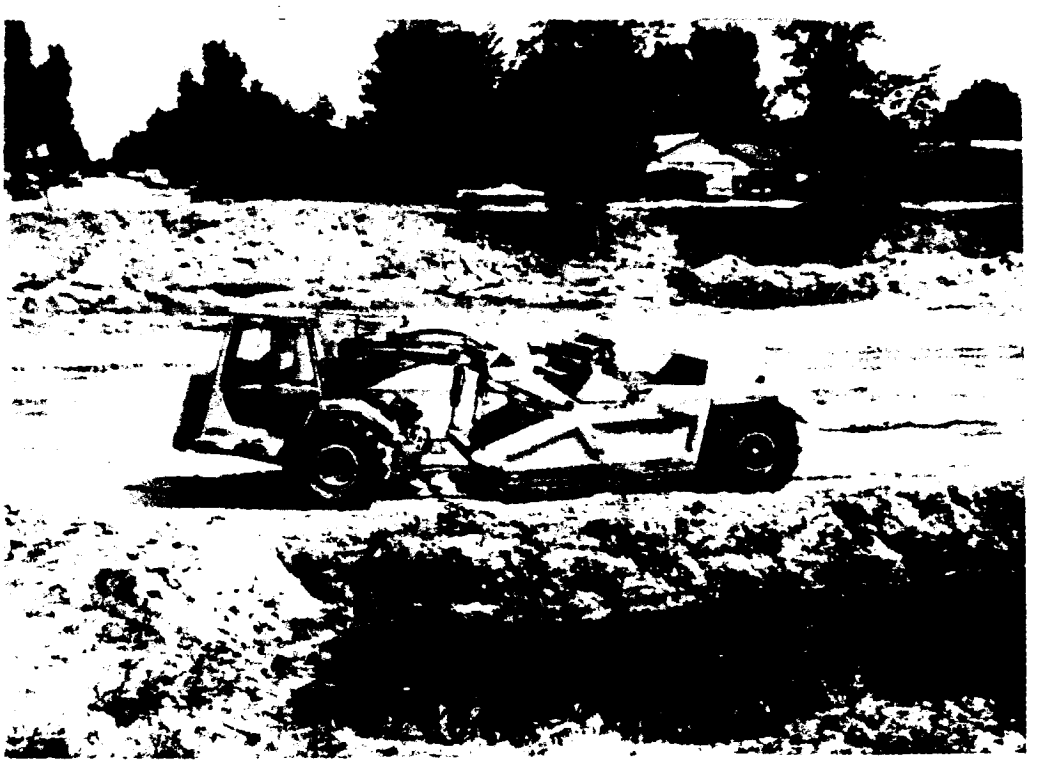
For large open channels, large diameter pipes, box culverts or extremely deep storm sewer construction, it may be practical and feasible to excavate with earthmoving equipment. This procedure involves operations very similar to those used in grading or embankment construction. Large scrapers may be used to bring the excavation down to a level where more precise excavation is practical. Other alternative methods such as power shovels, draglines, and belt loaders, in conjunction with specialized hauling equipment, may be used in lieu of scrapers. Such equipment may be preferred under certain combinations of excavated materials, haul routes and haul distances.

In some cases, it may be necessary to distinguish between general soils and firmer materials, such as rock, which may require some preliminary breaking up prior to excavation. In this case different procedures must be established for handling the material.

Because of the large volume of materials generally excavated by this method, it is often necessary to carefully evaluate stockpile locations, haul lengths, and haul routes. This will often be the contractor's responsibility and should be addressed during the bidding and preconstruction phases of the work. After award of the contract, the engineer should provide assistance to the contractor in locating stockpiles and in identifying materials suitable for backfill.

D. Small-Scale Excavation

Where large earthmoving equipment is not appropriate and trenching operations are also not suitable, smaller excavation equipment may be useful. This type of application might include the installation of culverts, manholes, unusual channel structures or special construction encountered when using earthmoving equipment.



Self-loading scraper being used for channel excavation.

The type of equipment most often used are backhoes, excavators, front-end loaders,and bulldozers. The preferred equipment is most often dictated by type of facility and nature of material to be excavated. Rubber-tired backhoes are frequently used when the extent of excavation is small. Tracked backhoes or excavators allow excavating without the use of stabilizing legs and can be used to great advantage for trenching. Culvert crossings, manholes and junction structures, and areas with limited access are situations where excavators are most frequently used.

These machines are available with bucket capacities varying from ⅜–3 cubic yards and more. They are convenient for the excavation of trenches with widths exceeding 2′ and to depths down to 25′. They are the most satisfactory equipment for excavation in loosened rock.

The excavator may also be used with a cable sling for lowering sewer pipe into the trench. Where safety requirements and the soil do not require sheeting and bracing, this method is very economical. When sheeting and bracing must follow the excavation closely, the use of an excavator for excavation and a crane for placement of sewer pipe is a common practice.

Front-end loaders are commonly used where there is a relatively wide area that requires excavation, but the excavation volume is not large. This might be the case in the installation of box culvert road crossings

or short segments of large diameter conduits or open channels where the use of large earthmoving equipment is not justified.

In wide, deep trenches, the front-end loader has sometimes been used as an auxiliary to an excavator or clamshell. In this arrangement the loader excavates the upper part of the trench, leaving the bottom bench or benches for the excavator or clamshell that completes the excavation.

The principal use of loaders in many storm sewer construction projects, however, is in transporting sewer pipe, manhole sections, and other appurtenant structures and granular bedding and backfill material to the trenches.

Bulldozers are generally useful when rippable rock is encountered on the site. The bulldozer can be used to fracture the rock and then move it short distances away from the excavation. In general, it will not be practical to use bulldozers to move material any significant distance. In these cases front-end loaders and trucks will be required to transport the material to a suitable stockpile or disposal site.

E. Dredging

Dredging is an operation commonly performed beneath the water table, through open water or in stream crossings. When it is infeasible to de-water the area, construction by dredging may be required. This generally involves the use of draglines.

The dragline is also useful under some other circumstances. In open country for stream crossings or in a wide right-of-way, it may be feasible to do a large part of the excavation by means of a dragline, allowing the sides of the trenches to acquire their natural slope. In cases of very deep trench excavation, on the order of 30–50′, the dragline has been used for the upper part of the excavation, with a backhoe operating at an intermediate level. By rotating the backhoe, the material thus excavated can be relayed to the dragline, which then lifts it to the spoil bank or to trucks at the surface.

F. Rock Excavation

When rock cannot be ripped with bulldozers or other mechanical equipment, drilling and blasting must be used. In addition, some shales and softer rocks, which may be ripped in open excavation, will require blasting before they can be removed in confined areas.

Normally the most economic method will involve pre-shooting; that is, drilling and shooting rock before removal of overburden. In some instances the presence of wet granular materials above the rock ledge will necessitate stripping before drilling, since holes cannot be kept open through the overburden to permit placing of explosive charges.

For narrow trenches in soft rock, a single row of drill holes may be sufficient. One or more additional rows may be required in harder rock, or for wider trenches. To reduce overbreak and improve bottom fragmentation, time delays should be used in blasting for trenches. In tight

quarters, trench walls can be pre-split, with the material in between blasted in successive short rounds to an open face to produce minimum vibration.

It must be recognized that there will be a minimum feasible trench width varying with the rock formation, and in the case of small storm sewers, it may be necessary to design the conduit for the extreme loads of the positive projecting condition.

All ground and air pressures that result from blasting should be recorded on a sealed cassette seismograph. Surveys of adjacent structures for the presence of cracks before blasting commences should be considered. Blasting should be done only by persons experienced in such operations.

VI. OPEN-TRENCH CONSTRUCTION

A. Trench Dimensions

As noted in Chapter 14, the trench at and below the top of the storm sewer should be only as wide as necessary for proper installation and backfilling, consistent with safety requirements. The contract must provide for alternate methods or require corrective measures to be employed by the contractor if allowable trench widths are exceeded through overshooting to rock, caving of earth trenches, or over-excavation. The width of trench from a plane 1 foot above the top of the storm sewer to the ground surface is related primarily to its effect on the safety of the workmen who must enter the trench and on adjoining facilities, such as other utilities, surface improvements, and nearby structures.

In undeveloped subdivisions and in open country, economic considerations often justify sloping the sides of the trench for earth stability from a plane 1′ above the top of the finished storm sewer to the ground surface. This eliminates placing, maintaining, and removing substantial amounts of temporary sheeting and bracing, although safety regulations may make some type of sheeting or bracing mandatory.

In improved streets, on the other hand, it may be desirable to restrict the trench width to protect existing facilities and reduce the cost of surface restoration. Available working space, traffic conditions, and economics will all influence this decision.

B. Excavation Procedures

The method and equipment used for excavating the trench will depend on the type of material to be removed, the depth, the amount of space available for operation of equipment and storage of excavated material, and prevailing practice in the area. Ordinarily the choice of method and equipment rests with the contractor. However, various types of equipment have practical and real limitations regarding minimum trench widths and depths. The contractor is obligated, therefore, to use only that equipment capable of meeting trench width limitations

imposed by pipe strength requirements, or for other reasons set forth in the technical specifications.

Spoil should be placed sufficiently back from the edge of the excavation to prevent caving of the trench wall and to permit safe access along the trench. With sheeted trenches, a minimum distance of 3 feet from the edge of the sheeting to the toe of the spoil bank will normally provide safe and adequate access. Under such conditions the supports must be designed for the added surcharge. In unsupported trenches the minimum distance from the vertical projection of the trench wall to the toe of the spoil bank normally should be not less than one-half the total depth of excavation. In most soils, this distance will be greater in order to provide safe access beyond the sloped trench walls.

Trenching machines are machines generally used for shallow trenches less than 5′ deep. For installation of small storm sewers in cohesive soils, the trenching machine can make rapid progress at low cost.

When the protection of other underground structures or soil conditions require close sheeting and the use of vertical-lift equipment, the clamshell bucket is used. In very deep trenches where two-stage excavation is required, the backhoe is sometimes used in combination with the clamshell, with the backhoe advancing the upper part of the excavation and the clamshell following for the lower. Sheeting and bracing of the upper part are installed as required prior to the excavation of the lower part, and the installation of the lower-stage of sheeting.

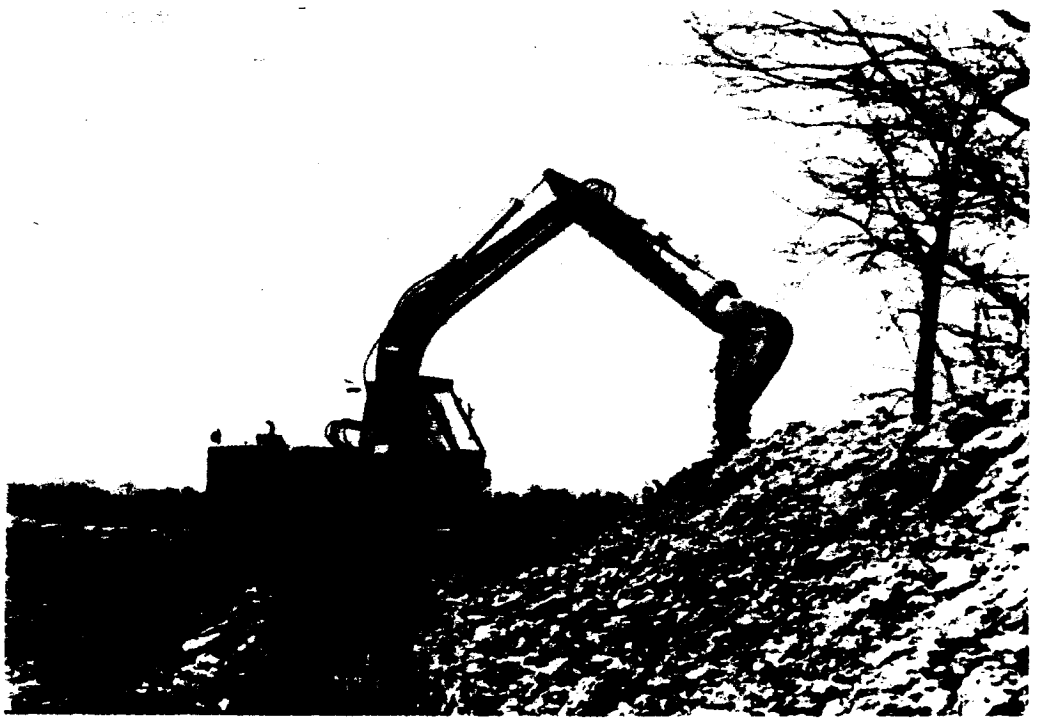
C. Sheeting and Bracing

Trench sheeting and bracing should be adequate to prevent cave-in of the trench walls or subsidence of areas adjacent to the trench, and to prevent sloughing of the base of the excavation from water seepage. Contracts normally stipulate that the contractor is responsible for the adequacy of any required sheeting and bracing. The strength design of the system of supports should be based on the materials encountered. Sheeting and bracing always must comply with applicable safety requirements.

For wider and deeper trenches a system of wales and cross struts of heavy timber (or steel sections) often is used. Sheeting is installed outside the horizontal wales as required to maintain the stability of the trench walls. Jacks mounted on one end of the cross struts maintain pressure against the wales and sheeting.

In some soil conditions it has been found economical and practical to use steel trench shields that are pulled forward as sewer pipe laying progresses. Care must be exercised in pulling shields forward so as not to drag or otherwise disturb the previously laid pipe sections or to create conditions not assumed in calculating trench loads.

In non-cohesive soils containing considerable groundwater, it may be necessary to use continuous steel sheet piling to prevent excessive soil movement. Such steel piling sometimes extends several feet below the bottom of the trench unless the lower part of the trench is in firm material.



Excavator being used in trenching operation.

In some soils, steel sheet piling can be used with a backhoe operation for the upper part of excavation, but the piling usually needs to be braced before the excavation has reached its full depth. The remaining excavation is performed by vertical-lift equipment such as a clamshell.

Another means of trench sheeting occasionally adopted involves the use of vertical H-beams as "soldier beams" with horizontal wooden lagging. This is sometimes advantageous for trenches under existing overhead viaducts where overhead clearances are low and spread footings lie alongside the trench walls. The vertical beam can be tilted and driven. As excavation progresses downward, the lagging is installed between adjacent pairs of soldier beams. For deep trenches with limited overhead clearances the soldier beams can be delivered to the site in shorter lengths and their ends field-welded as driving progresses.

The removal of sheeting following pipe laying may affect the load on the pipe or adjacent structures (see Chapter 14). This possibility must be considered during the design phase. If removal is to be required or permitted, appropriate directions must be included in the technical specifications for proper removal of sheeting and placement of backfill to thoroughly fill the voids thus created.

VII. TUNNELLING

Tunnelling is considered to be any construction method that results in the placement or construction of an underground conduit without

Trench box used in trench excavation.

continuous disturbance of the ground surface, and includes the various forms of jacking of prefabricated units from shaft or pit locations. Tunnelling methods applicable to storm sewer construction can be classified generally as:

(a) Auger or boring method.
(b) Jacking of preformed steel or concrete pipe.
(c) Mining methods.

A. Auger or Boring Method

In sizes less than 36″ diameter, rigid steel or concrete pipe can be pushed for reasonable distances through the ground and the earth

removed by mechanical means under the control of an operator at the shaft or pit location. Several types of earth augers are available, and some contractors specialize in this type of operation. Augers as large as 72" have been used, but for sizes above 36" considerable care must be exercised to avoid overbreak. In the case of concrete pipe, it may be necessary to use an auger with a special head having a diameter equal to the outside diameter of the pipe being placed.

The presence of well-cemented soils is a serious deterrent to this method of installation. If such soils are expected, particularly when sewer pipes smaller than 36" are to be placed, it may be more economical first to install an oversize lining by conventional tunnel or jacking methods. The sewer pipe then can be placed within the liner pipe and the remaining space backfilled with sand, cement grout, or concrete.

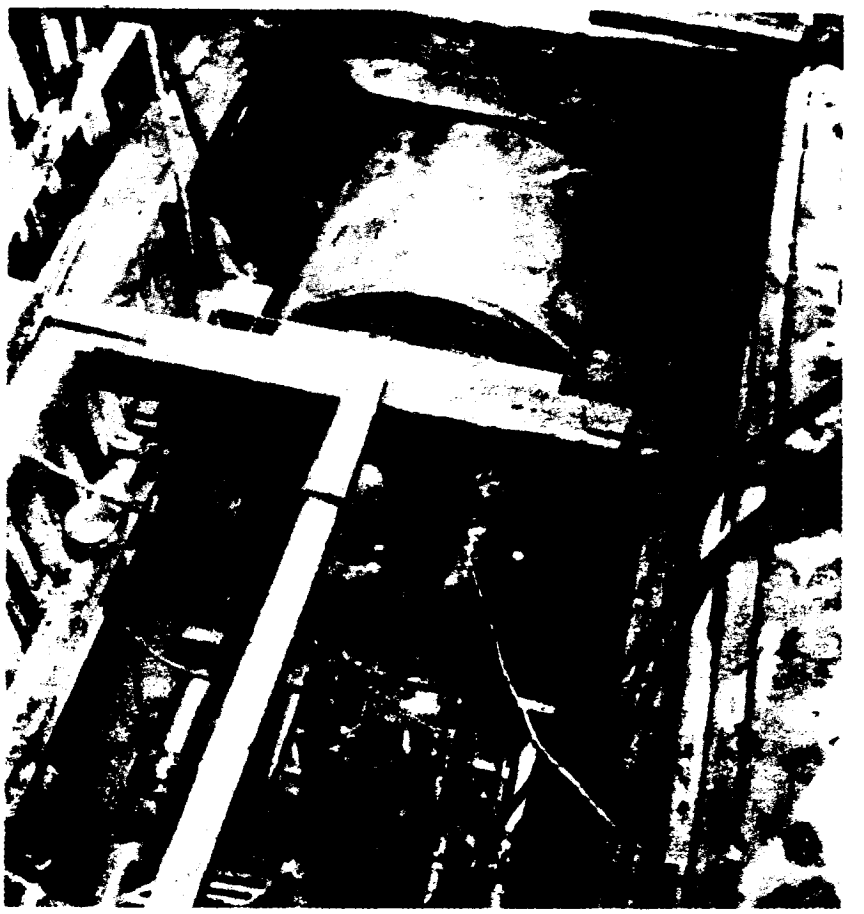
B. Jacking

Although the limits will vary with geographic locations and soil conditions, finished interior diameters of 30–108 inches are the generally accepted limits for pipe jacking. Excavation and removal of the excavated material is done by machine or manually, augmented with air spades, special knives, etc. The most commonly used materials for such jacking operations are reinforced concrete or smooth steel pipe. The pipe selected for jacking must be strong enough to withstand the loads exerted by the jacking process.

The usual procedure is to equip the leading edge with a cutter or shoe to protect the sewer pipe. As succeeding lengths of pipe are added between the leading sewer pipe and the jacks and the sewer pipe jacked forward, soil is excavated and removed through the sewer pipe. Material is trimmed with care and excavation does not precede the jacking operations more than necessary. Such a method usually results in minimum disturbance of the natural soils adjacent to the sewer pipe.

When jacking, contractors have sometimes found it desirable to coat the outside of the pipe with a lubricant, such as bentonite, to reduce frictional resistance. In some instances this lubricant has been applied through pressure fittings installed in the wall of the leading pipe. Grout holes sometimes are provided in the walls of the pipes for use in filling outside voids. Protective joint spacers are used to prevent damage to pipe joints. Because soil friction may increase with time, it is desirable to continue jacking operations without interruption until completed.

In all jacking operations it is important that the direction of jacking be carefully established prior to the start of work and checked periodically during the work. Guide rails must be installed in the bottom of the jacking pit or shaft. In the case of a large pipe it is desirable to have such rails carefully set in a concrete slab. The number and capacity of the jacks used depend primarily on the size and length of the pipe to be placed and the type of soil encountered. Backstops must be strong enough and large enough to distribute the maximum loading of the jacks to the soil behind them.



Jacking large-diameter reinforced concrete pipe under a railroad.

In some cases long sewer lines have been installed by jacking from a series of shaft locations spaced along the line of the sewer pipe.

C. Mining Methods

Tunnels with finished interior dimensions of 5′ or larger in clay or granular materials ordinarily are built either with the use of tunnel shields or with boring machines, or by open-face mining with or without some breasting. Rock tunnels normally are excavated open-face by conventional mining methods or with boring machines.

1. Tunnel Shields

In clays, silts, sands and gravels, especially in built-up city areas, it will usually be necessary to use tunnel shields for tunnelling operation. Compressed air also may be required to control the entry of water into the tunnel if the phreatic line is above the tunnel invert and the soils lack adequate cohesion.

With a shield it is necessary to install a primary lining of sufficient strength to support the surrounding earth and to provide a progressive backstop for the jacks that advance the shield. The lining may be installed against the earth and the annular space between the lining and the earth filled with pea gravel and grout. Alternatively, the tunnel lining may be expanded against the earth as the shield is advanced. The latter method practically eliminates the need for grouting the annular opening.

2. *Boring Machines*

Tunnel boring machines, also called digger shields or mechanical moles, have been developed for tunnel excavation in clay and rock. They usually have cutters mounted on a rotating head which is advanced into the heading. A conveyor system moves muck away from the tunnel face. Machines may be braced against the walls of the excavation or against previously placed tunnel lining. Some machines also are equipped with shields. Machines have been used successfully in the construction of tunnels in clay up to 25′ in diameter and in rock up to 36 feet in diameter. Machines are most useful in fairly long runs through generally similar material.

3. *Open-Face Mining Without Shields*

Where the ground allows the use of open-face mining methods it is often more economical to use segmental supports of wood or steel for the sides and top of the tunnel only. The need for compressed air or breast boards in the tunnel heading will depend on the type of soil and amount of moisture or groundwater. The geotechnical report will generally be the best guide to geology and groundwater conditions to be encountered. The particular combination of geology and groundwater will determine the need to use compressed air.

4. *Primary or Temporary Lining*

Materials used for primary lining are usually steel, wood or a combination of the two. Linings also may be made of segmental precast concrete, stamped steel or cast iron.

Some engineers and contractors prefer to use continuous rings of liner plates having sufficient section modulus to resist the earth pressures without use of special structure ribs or rings. A circular lining formed of such plates becomes a compression ring and has some inherent stability not equalled by horseshoe-shaped supports. Soil conditions and the contractor's preference determine the choice of such a lining. If design of liner plate support is based on the assumption that plates will act as a compression ring, immediate grouting behind liner plates or immediate expansion of the lining is required to insure uniform loading. In any event, voids behind liner plates should be grouted or the lining expanded prior to subsidence of the overburden.

Tunnel lining used in open face mining operation.

5. *Tunnel Excavating Equipment*

The type of excavating equipment or tools used in tunnelling depends on the kind of material to be excavated and the work space available. Pneumatic spades and special knives are used widely in excavating clay. Drilling and blasting are usually employed in rock tunnels. In the case of shale, roadheaders and undercutting machines like those used in coal mining have been used to advantage.

6. *Shafts*

Where tunnels are of considerable length, one or more construction shafts may be necessary. On important thoroughfares these shafts are better located in an adjacent side street or vacant lot, with access to the work provided through a short connecting entry tunnel.

Offset shaft locations are especially desirable when soil conditions require the use of compressed air. In such a case only one air lock in the entry tunnel will be required. Shafts generally are located so that tunnelling in both directions is possible. Construction shafts on long tunnels typically are spaced 1,200–2,500 feet apart. Factors tending to affect this spacing are the need for compressed air, the size of tunnel, and the depth below ground.

Shafts should be large enough to permit the installation of an electric hoist. Such equipment should be used only for the handling of material,

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with separate personnel lifts in deep shafts. Hoisting in shafts by means of a crane may be permitted when the length of the tunnel is short and safety precautions are taken to prevent engine exhaust from entering the shaft and tunnel.

Tunnel drainage is normally discharged from the shaft, and the shaft must be equipped with some form of collection sump and drainage pump.

7. Compressed Air Equipment and Locks

Compressed-air equipment for tunnelling should have sufficient capacity to maintain a pressure that will balance the hydrostatic pressures in the soil at tunnel depth.

The equipment includes compressors, air receivers, piping, control valves, air locks, main and emergency locks, bulkhead walls, gages, etc. Separate locks should be provided for materials and personnel. Generally for long, large diameter tunnels, electrically operated compressors are used with two independent sources of power. Standby compressors in many cases are either diesel or gasoline powered.

8. Ventilating Air

In compressed air tunnels, air must be circulated in sufficient quantity to permit the work to be done without danger or excessive discomfort.

In free air tunnels, the ventilation rate must be adequate to clear the tunnel of gases in a maximum of 15 minutes if explosives are used. Rates also must be adequate to dilute exhaust of permissible diesel equipment to safe limits. A minimum of 200 cubic feet/minute of fresh air per employee underground should be provided. In cold weather it may be necessary to condition ventilating air to prevent excessive fogging at the heading. Air should be monitored constantly for toxic or flammable gases and airborne contaminants. A record of all tests should be maintained.

VIII. DEWATERING

Storm sewers or other storm drainage systems generally lie in or near the lowest point of the drainageway. This provides for the optimum drainage to the facilities, but also can cause problems during construction. Handling flows entering the construction area is frequently one of the biggest problems confronting the contractor during drainage improvement projects. The contractor is generally solely responsible for the dewatering of the work area; however, it is frequently advisable for the engineer to review the contractor's dewatering plan.

Dewatering of excavations and trenches is necessary to provide proper working conditions for the construction or installation of storm sewers. Where possible, the most economical means is the diversion of up-

stream inflows. This allows for the elimination of flow before it enters the excavation. This can be accomplished by physical diversion of the inflow through cutoffs and diversion channels or pumping around the project site. Where upstream flows cannot be diverted, or where groundwater enters the excavation, it must be removed.

Excavations should be dewatered for concrete placement and sewer pipe laying, and they should be kept continuously dewatered for as long as necessary. Unfortunately, the disposal of large quantities of water from this operation, in the absence of existing storm drains or adjacent water courses, may present problems. The possibility of draining the water through the completed sewer to a permissible point of discharge may be considered when other means of disposal are unavailable. Sufficient precautions must be taken to prevent scour of freshly placed concrete or mortar.

Crushed stone or gravel, possibly in combination with a geotextile, may be used as a sub-drain to facilitate drainage to trench or sump pumps. It is good practice to provide clay dams in the sub-drain to minimize the possibility of excessive groundwater flows undercutting the sewer foundation.

An excessive quantity of water, particularly when it creates an unstable soil condition, may require the use of a well-point system. A system of this type consists of a series of perforated pipes driven or jetted into the water-bearing strata on either side of the sewer trench and connected to a pump by a header pipe. The equipment for a well-point system is expensive and specialized. General contractors often seek the help of special dewatering contractors for such work. Well-point systems must be run continuously to avoid disturbing the excavated trench bottom by uplift pressure.

When excavating in coarse water-bearing material, turbine well pumps may be used to lower the water table. Chemical or cement grouting and freezing of the soil adjacent to the excavation have been used in extremely unstable water-bearing strata.

Water from all types of dewatering systems should be checked periodically to assure that fine-grained material is not being removed from beneath the pipe. This might cause future pipe settlement.

Care must be exercised to insure that property damage, including silt deposits in sewers and on streets, does not result from the disposal of diverted drainage. The water control plan should be developed and implemented in a manner that does not impact the quality or quantity of water in downstream drainageways.

IX. FOUNDATIONS

The foundation of the storm sewer is critical to the structural integrity of the facility. Firm cohesive soils provide adequate sewer pipe foundations when properly prepared. Occasionally the trench bottom may

be shaped to fit the sewer pipe barrel and holes dug to receive projecting joint elements. It is often a practice to over-excavate and backfill with granular material, such as crushed stone, crushed slag, or gravel, to provide uniform bedding of the sewer pipe. Such granular bedding is used because it is both practical and economical.

In very soft bottoms it is frequently necessary to first overexcavate to greater depths and stabilize the trench bottom by the addition of gravel or crushed slag or rock compacted to receive the load. The stabilizing material must be graded to prevent movement of the subgrade up into the stabilizing base, and the base into the bedding material. There is increasing use of specialized filter fabrics to prevent this movement. The required stabilization depth should be determined by tests and observations on the job.

Where the trench bottom cannot be stabilized satisfactorily with a crushed rock or gravel bed, and where limited and intermittent areas of unequal settlement are anticipated, a timber cribbing, piling, or reinforced concrete cradle may be necessary.

Where the bottom of the trench is rock, it must be overexcavated to make room for an adequate bedding of granular material, which will uniformly support the conduit. The trench bottom must be cleaned of shattered and decomposed rock or shale prior to placement of bedding.

In some instances, a sewer pipe must be constructed for considerable distances in areas that are subject to subsidence. If the subsidence is shallow, consideration should be given to constructing the sewer on a timber platform or reinforced concrete cradle supported by piling. The sewer's support should be adequate to sustain the weight of the full sewer and backfill. Piling in this case is sometimes driven to grade with a follower prior to making the excavation. This practice avoids subsidence of trench walls resulting from pile driving vibrations. Extreme care must be taken to locate all underground structures.

X. BACKFILLING

Backfilling is an important consideration in construction. The methods and equipment used in placing fill must be selected to provide the appropriate character and compaction of the fill. The method of backfilling varies with the width of the excavation, the character of the materials excavated, the method of excavation, and the degree of compaction required.

A. Degree of Compaction

For trench backfill in improved streets or streets programmed for immediate paving, a high degree of compaction is normally required. In less important streets or in sparsely inhabited subdivisions where flexible macadam roadways are used, a more moderate specification for backfilling may be justified. Along sewers in open country, it may be

sufficient to mound the trench and, after natural settlement, return to regrade the area. For general backfill of channel slopes or structural backfill, compaction should be based on structural criteria. The stability of slopes is often dependent on the degree of compaction. Structures often require a specified degree of compaction to satisfy structural design assumptions.

The degree of compaction required is generally expressed as some relation to the laboratory maximum density at optimum moisture content. The two most common tests used to determine maximum density of cohesive soils are ASTM D-698 (standard Proctor density) and ASTM D-1557 (modified Proctor density). For cohesionless soils, ASTM D-4253 and D-4254 are more appropriate tests to determine the maximum density and optimum moisture.

The field density is generally measured by the sand-cone method (ASTM D-1556), the balloon method (ASTM D-2167), or nuclear density meter (ASTM D-2992 and D-3017). For cohesive soils, the degree of compaction required should generally be expressed as the relative compaction. This is defined as the ratio of the field dry density to the laboratory maximum dry density and is expressed as a percentage. For cohesionless free-draining soils that do not exhibit a well-defined moisture/density relationship under impact compaction, vibrating compaction is more appropriate. The degree of compaction required for these soils is expressed as the relative density and is determined by the relationship between the field density, the minimum density, and the maximum density.

$$\frac{D_{max}(D_{field} - D_{min})}{D_{field}(D_{max} - D_{min})} \times 100 \qquad (16\text{-}1)$$

B. Trench Backfilling Sequence

Backfilling should proceed immediately on curing of trench-made joints and after the concrete cradle, arch, or other structures gain sufficient strength to withstand loads without damage. In areas of construction not requiring work beyond pipe placement, backfilling should immediately follow pipe placement.

Backfill generally is specified as consisting of three zones with separate and distinct criteria for each: The first zone (pipe zone) extends from the foundation material to 12″ above the top of the sewer pipe or structure; an intermediate zone generally contains the major volume of the fill; and the upper zone consists of pavement subgrade, finish grading materials, topsoil, etc.

The first zone should consist of selected materials placed by hand or by suitable equipment in such a manner as not to disturb the sewer pipe, and compacted to a density consistent with design assumptions. In some instances the material used for granular bedding is brought

above the sewer to insure high density backfill with minimum compactive effort. When installing flexible pipes, attention must be given to proper placement and compaction of the haunching material from the base of the pipe to the springline. When high water tables are anticipated, backfill materials without substantial voids are required to prevent soil migration.

Compaction of the intermediate zone is usually controlled by the location of the trench. Under traffic areas or other improved existing surfaces, a high degree of compaction may be required. In undeveloped areas, little compaction may be required. In general, the degree of compaction required will affect the choice of material. The use of excavated material, if suitable, is usually desirable in areas subject to frost heave so that excavated areas will move no more and no less than undisturbed areas.

Depth and compaction of the upper zone are dependent on the type of finish surface to be provided. If the construction area is to be seeded or sodded, the upper 18″ may consist of 14″ of select material slightly mounded over the trench and lightly rolled, covered by 4″ of top soil. If the area is to be paved, the upper zone must be constructed to the proper elevation for receiving base and paving courses under conditions matching design assumptions for the subgrade. If the trench backfill is completed in advance of paving, the top 6″ of the upper zone should be scarified and recompacted prior to paving. In such cases, it may be necessary to install a temporary surface to be replaced at a later date with permanent pavement.

Before and during the backfilling of an excavation, precautions should be taken to prevent flotation of pipelines due to the entry of large quantities of water into the trench. The buoyant forces may affect the vertical alignment of the pipe. A check of the hydrostatic pressures causing uplift is advisable for sewers in areas of high groundwater.

There are cases where pipe is laid without excavation (there is no trenching—rather the pipe is laid on a prepared base and fill material is then placed around the pipe). Such pipes frequently are of large diameter, and particular attention must be given to backfilling in the haunch areas and beside the pipe to provide lateral support to prevent compression failures. Construction loads in these cases can far exceed normal service loads.

In all cases, the level of compaction required is dictated by the design assumptions with respect to pipe and structure foundations, the location of the excavation, and the character of backfill materials. A thorough geotechnical analysis should be performed, and the results used to determine the final degree of compaction necessary to achieve the proper installation.

C. Methods of Compaction

Cohesive materials with high clay content are characterized by small particle size and low internal friction. They have small ranges of mois-

ture content over which they may be compacted satisfactorily and are very impervious when compacted. Because of the strong adhesion of the soil particles, high pressures must be exerted to shear the adhesive forces and remold the particles into a dense soil mass. This dictates the use of impact-type equipment for most satisfactory results. In confined areas, pneumatic tampers and engine-driven rammers may give good results. The upper portion of narrow trenches can be consolidated by self-propelled rammers. In wide excavations sheeps-foot rollers may be used; if the degree of compaction required is not high, bulldozers and loaders may be used to compact the fill.

Regardless of equipment used, the soil must be near optimum moisture content and compacted in multiple lifts generally not exceeding 8″ in loose depth if satisfactory results are to be obtained. The trench bottom must be free of excessive water before the first lift of backfill is placed.

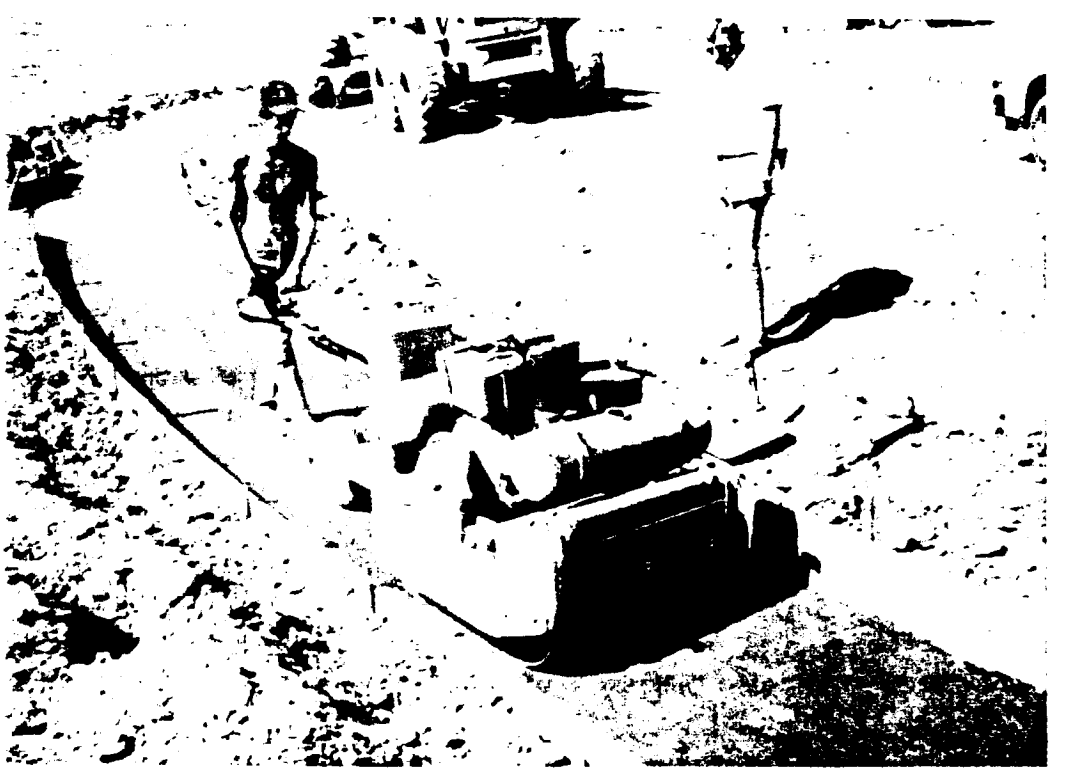
If the material has a high moisture content at the time of excavation, some preparation of the material probably will be required before spreading in the trench. This may include pulverizing, drying, or blending with dry or granular materials to improve placement and consolidation.

Tamping foot self-propelled compactor compacting backfill.

As noted earlier, cohesionless materials are best compacted using vibratory equipment. Moisture content at the time of compaction is not so critical, and consolidation is effected by reducing the surface friction between particles thus allowing them to rearrange in a more compact mass. In confined areas, vibratory plates give the best results. For wider excavations vibratory rollers are most satisfactory. Again, if the degree of compaction required is not high, and if layers are thin, the vibration imparted by dozer or loader tracks may result in satisfactory consolidation.

In some areas water is used to consolidate granular materials. Unless the fill is saturated and immersion vibrators are used, the degree and uniformity of compaction cannot be controlled closely. With some materials, adequate compaction may be obtained by draining water used to saturate or puddle fill through drains constructed in structure walls. These drains are capped after the backfill has drained.

Sometimes the material removed from the excavation may be entirely unsatisfactory for backfill. In this case, selected materials must be hauled in from other sources. Cohesive materials, noncohesive materials, or a combination of these may be used, but an assessment must be made of the possible change in groundwater movement that the use of outside



Vibratory steel drum compacting subgrade of low flow channel.

materials may cause. For example, the use of cohesive materials to backfill a trench in rock could result in a dam impervious to groundwater travelling in rock faults, seams, and crevices. On the other hand, granular materials placed in a clay trench could result in a very effective sub-drain.

XI. PIPE STORM SEWERS

A. Storm Sewer Pipe Quality

Storm sewer pipe inspection is properly conducted by the manufacturer and by independent testing and inspection laboratories. Moreover, with pipe storm sewers, transportation charges may constitute a substantial portion of material costs, and as a result inspection at the pipe plant is usually desirable. Inspection may consist of visual inspection of workmanship, surface finish, and markings; physical check of length, thickness, diameter, and joint integrity and tolerances; proof of crushing strength (rigid pipe) or pipe stiffness (flexible pipe), design materials tests, and tests of representative specimens. If three-edge bearing tests are not used on precast concrete pipe, core or cylinder tests should be required. Standard cylinder tests are not practical with the mixes used in some manufacturing methods and core tests are generally used. Cores also permit checking tolerances on placement of reinforcing cages.

Storm sewer pipe suppliers should furnish certificates of compliance with specifications that can be easily checked as the storm sewer pipe arrives at the site. Storm sewer pipe also should be checked visually at time of delivery for possible damage in transit, and again as it is laid for damage in storage or handling.

B. Storm Sewer Pipe Handling

Care must be exercised in handling and bedding all precast storm sewer pipe, regardless of cross-sectional shape. All phases of construction should be undertaken to insure that, insofar as practical, pipe is installed as designed. Pipe should be handled during delivery in a manner that eliminates any possibility of high impact or point loading due to dropping or impacting, with care taken always to protect the joints.

C. Storm Sewer Pipe Placement

Storm sewer pipe should be laid on a firm but slightly yielding bedding, true to line and grade, with uniform bearing under the full length of its barrel, without break from structure to structure, and with the socket ends of bell and spigot or tongue and groove storm sewer pipe joint facing upgrade. Storm sewer pipe should be supported free of the bedding during the jointing process to avoid disturbance of the subgrade. A suitable excavation should be made to receive sewer pipe bells and

Installation of large diameter storm sewer.

joint collars where applicable so that the bottom reaction and support are confined only to the pipe barrel. Adjustments to line and grade should be made by scraping away or adding adequately compacted foundation material under the pipe and not by using wedges and blocks or by beating on the pipe.

Extreme care should be taken in jointing to insure that the bell and spigot are clean and free of any foreign materials. Joint materials vary with the type of storm sewer pipe used. All pipe joints should be made properly using the jointing materials and methods specified. All pipe joints should be sufficiently tight to meet infiltration or exfiltration tests.

In large diameter storm sewers with compression-type joints, considerable force will be required to insert the spigot fully into the bell. Come-alongs and winches or the crane itself may be rigged to provide the necessary force. Inserts should be used to prevent the storm sewer pipe from being thrust completely home prior to checking gasket location. After the gasket is checked, the inserts can be removed and the joint completed.

The operation of equipment over small diameter storm sewer pipe, or other actions that would otherwise disturb any conduit after pipe jointing, must not be permitted.

At the close of each day's work, or when storm sewer pipe is not being laid, the end of the pipe should be protected by a close-fitting stopper to keep the pipe clean and to prevent unwanted access into the pipe, with adequate precautions taken to overcome possible uplift. The elevation of the last storm sewer pipe placed should be checked the next morning before work resumes.

If the storm sewer pipe load carrying capacity is increased with either arch or total encasement, contraction joints should be provided at regular intervals in the encasement coincident with the pipe joints to increase flexibility of the encased conduit.

D. Manholes and Inlets

The two primary appurtenances to pipe storm sewer construction are manholes and inlets. These appurtenances are essential to the proper functioning of storm sewer systems. The materials most commonly used for manhole and inlet construction include precast concrete sections and cast-in-place concrete.

Proper construction methods are important in the installation of manholes and inlets and, as with the storm sewer itself, proper backfill compaction is necessary.

XII. OPEN CHANNELS

The excavation for open channels may be accomplished by various types of earth-moving equipment, depending on the size of the facility. Small channels may be excavated to grade by backhoes or excavators whereas large channels may be excavated by large earth-moving equipment.

A. Trapezoidal Channels

Trapezoidal channels are normally lined with grass, concrete, riprap or gabions. A trapezoidal channel that is to be lined with concrete should be excavated to very close tolerances to control the amount of concrete. A grass-lined channel normally will have topsoil placed on it to bring it to grade. Riprap or gabion lined channels need to be over-

excavated to allow for the design thickness of the riprap or gabions being used.

Where a concrete lining is used, the bottom is usually a poured and screeded slab. Care must be taken with the placement of concrete to assure that reinforcing steel or wire mesh is properly positioned. Some concrete channels can be constructed with special slip form pavers, or using pre-cast units. In the latter case, the bottom is usually a poured slab with a stub and key-way for the wall to rest on. Any impervious lining should be provided with weep holes for pressure equalization.

B. Rectangular Channels

Rectangular channels are usually constructed where space is limited. The type of rectangular structure will dictate which construction method is to be used. Space limitations, cost, and visual appearance often dictate the materials used for the rectangular channel. Alternative materials to cast-in-place concrete for wall construction are often timber or metal cribbing, soil cement, rock gabions, reinforced earth, precast concrete, or sheet piling. Where cast-in-place concrete is used, the quantity, placement, and cleanliness of the reinforcing steel must be carefully checked.

C. Low-Flow Channels

In conjunction with either rectangular or trapezoidal channels, low-flow channels (or trickle channels) are often constructed to contain the base flows within the bottom of the channel. These channels are typically lower in elevation than the primary channel and, as a result, may be able to provide drainage for the subgrade beneath the structures.

D. Structures

Open channel storm drainage systems often include appurtenances such as drop structures, culvert crossings and siphons. During the excavation for structures, a geotechnical engineer should be available for inspection and recommendations regarding the foundation material and suitability of the excavated material for backfill. If the foundation material is inadequate for the bearing pressure required, over-excavation and replacement with satisfactory materials may be necessary. Backfilling should be done in a manner that will not block weep holes. To prevent plugging, a granular filter, sometimes with a geotextile covering, should be placed behind the weep holes.

1. Drop Structures

Drop structures should be constructed with care to assure that flows will not undermine the structure. They may be constructed of concrete, riprap, gabions, sheet piling, or some combination thereof. A concrete structure must follow good structural construction practices with ade-

quate control of the foundation material and placement of the steel and concrete. Cutoff or crest walls are normally constructed along the length of the structure perpendicular to the flow to prevent flows from undermining the structure. Proper bedding beneath the structure is important to provide an adequate foundation for construction as well as to control subsurface water.

Riprap or gabion placement and grading must be controlled so that the design integrity is not sacrificed. Placement of riprap or gabions below the drop structure is often necessary to provide additional energy dissipation of the high flow velocities. Care should be taken to use adequately sized riprap or gabions to withstand these velocities.

2. *Culverts and Siphons*

As with drop structures, care needs to be taken with the construction of culverts and siphons to assure structural integrity, including adequate foundation and bedding. Once again, a geotechnical engineer should be available for inspection and recommendations regarding the foundation material and suitability of the excavated material for backfill.

Headwalls and wingwalls are structural components that require careful construction. Warped wingwalls are often provided in conjunction with culverts or siphons in open channels to provide a smoother transition of flow. Care must be taken when backfilling these walls as they vary in section from a vertical wingwall to a sloping concrete lining.

Debris walls are often constructed in conjunction with box culverts, bridges and siphons for the purpose of preventing the accumulation of debris from plugging or damaging the structure. Debris walls normally consist of an extension upstream of the concrete wall or pier dividing the cells of the box culvert, bridge, or siphon. These walls will of course be parallel to the direction of flow. Care must be exercised to provide a hardened surface (often steel) along the projecting face. The structural integrity of these walls is critical to ensure that the wall can withstand vibration and the forces acting upon it by accumulated debris and high flows.

XIII. RIPRAP

Riprap is one of the most common materials used for erosion protection in earth-lined channels. Riprap protection can be classified into two basic types, grouted and non-grouted:

A. Non-Grouted Riprap

Non-grouted riprap or dumped riprap is placed by means of a backhoe, excavator, or loader and is not bonded together by any artificial means, such as concrete, grout, or shotcrete. Therefore, to assure that

an area of riprap will retain its structural integrity, the rock must be properly graded. If an adequate amount of smaller rocks is not provided in the riprap, voids may result, which will create an unstable mass that may be subject to failure. Dumped riprap is not recommended on banks with a slope steeper than 2:1.

The type of rock or riprap installed should be carefully reviewed before installation. It is often advisable for the engineer to visit the quarry prior to commencement of construction operations. The rock should be quarried rock that is fractured and has a specific gravity of at least 2.5. If cobbles are used instead of fractured rock, they may lack the capability of interlocking together to make the solid mass required. Angularity enhances the interlocking capabilities and is normally specified.

Riprap with a mean size of less than nine inches may be installed, covered with soil and then seeded to create a more natural appearance. This type of installation does not require bedding beneath the layer of riprap. Riprap with a mean size greater than 9" usually requires bedding (0.75–1.5 inch crushed rock). The bedding allows free passage of water through the rock without allowing it to wash out the fine-grained materials beneath the riprap layer. If site conditions warrant, a layer of filter material may be placed beneath the bedding to further decrease the leaching of fines from the soil. The filter material may be either a geotextile or a properly designed granular filter. If a geotextile is used, the bedding material should be placed with sufficient care to protect the geotextile from ultraviolet light exposure and damage during riprap placement.

When placing riprap, great care should be taken that the riprap is properly toed into the channel bottom. The toe or bottom of the riprap blanket should be below the elevation of ultimate degradation or at least a depth equal to the thickness of the riprap layer below the channel bottom. If the riprap is not toed in, the flow of water in the stream may undermine the riprap, leading to a failure of the entire protected bank.

If a smoother surface is desired, it may be necessary to densify and smooth the riprap layer. This may be accomplished by rolling or plating the riprap. Plating riprap involves slapping the surface with a piece of steel plating (weighing approximately 5,000 pounds) or the back of an excavator bucket. The objective is to produce a reasonably smooth surface along the riprap. This practice will produce a tight, uniform blanket of rock with greater stability because of reduced drag on the individual stones and an increase in the angle of repose produced by the interlocking mass of rock. The smooth surface can also decrease the hydraulic roughness of the channel.

B. Grouted Riprap

Grouted riprap is bonded together as a single mass using concrete or grout. This type of riprap is generally used when the channel banks

are steeper than 2:1, when channel velocities are excessive, when turbulence is high, or when available stone is of insufficient size to satisfy hydraulic conditions.

As with dumped riprap, the gradation of the riprap for grouting is important, but for different reasons. The rocks should be large enough (mean size greater than 12 inches) so that the rocks will protrude high enough above the grout to effectively reduce the energy head of the water flowing in the channel. Also, the gradation should be such that the smaller rocks (less than six inches) are removed to allow the grout to fully penetrate the entire layer of the riprap.

Riprap that is to be grouted should be placed on bedding and possibly filter material. The bedding allows a free flow of water through the layer below the riprap. Weep holes should be installed to allow the ground water to be released from the bedding.

Full depth penetration of the grout is required to form a solid mass. To achieve this, several conditions are necessary. A high slump grout, as high as ten inches, should be used so that the grout flows into each void. During delivery of the grout, a low pressure pump with a minimum nozzle diameter nozzle of 2–4″ should be used to direct the grout

Grouting riprap with low-pressure grout pump.

to each void between the rocks at a manageable rate. When a small nozzle is used, smaller aggregate should be specified to prevent clogging. Some additives can also be included in the mix to increase its plasticity. If full depth penetration is still not achieved, then the grout may need to be vibrated.

Before grouting, the rocks should be washed to sluice the fines to the bottom of the riprap which will allow the grout to bond to the rocks. After placing the grout, a curing compound should be applied.

Some construction practices may be used to increase the aesthetic properties of the grouted riprap. Following the application of grout, it may be brushed with a small broom to give a smooth finish. Also, care should be taken during grouting to allow a minimum of grout to splash on top of the exposed rocks. Another method of improving the apppearance of the grouted riprap is to insert a dye into the grout to achieve a desired color.

XIV. EROSION AND SEDIMENT CONTROL

Clearing and stripping of land for storm drainage systems, if not properly conducted, may result in high localized erosion rates with subsequent deposition and damage to off-site properties. An erosion and sediment control plan can reduce the erosion and sediment deposition process to an acceptable level.

Several techniques (best management practices) are available for the control of erosion and sedimentation. Their effectiveness is often dependent upon the characteristics of the soils at the site as well as the topography, drainage, vegetation and other site features. Construction activities should be planned and completed in such a manner so that the exposed area of disturbed land is minimized, and so that the land is disturbed for the shortest possible period. Local regulations often detail acceptable practices for erosion and sediment control.

XV. SPECIAL CONSTRUCTION

A. Railroad Crossings

Storm sewers at times must be constructed under railroad tracks, which may be at street grade or on an existing railroad viaduct. Crossing of tracks at grade or on an embankment is usually accomplished most economically by jacking, boring, tunnelling, or a combination thereof. Usually, to satisfy railroad criteria, a casing pipe is installed and the storm sewer pipe is then placed inside.

When the distance from the base of rail to the top of the storm sewer is insufficient to allow jacking or tunnelling (usually less than one

diameter clearance), other construction means, which must be coordinated with the railroad, may be required. It may be necessary to provide a bypass and remove the affected tracks or, for local service lines, to remove the tracks and interrupt service during an open-cut operation. A temporary structure for support of the railroad tracks may also be a solution, with the storm sewer constructed in an open trench below that structure.

Construction of storm sewers under existing railroad viaducts involves a wide variety of methods, depending on the size of the storm sewer, its location in plan and elevation with respect to viaduct footing, type of footings, the nature of the soil, and the requirements of the railroad.

Where the soil is stable and the storm sewer is of sufficient size and is located satisfactorily with respect to viaduct footings, tunnelling may be both safe and economical. When the proposed storm sewer does not meet these criteria, close coordination with the railroad is essential and special methods of sheeting and bracing must be devised. To prevent subsequent movement of soil beneath the footings, all sheeting and bracing should be left in place.

In all cases, early planning with the railroad authorities is essential, since they generally have extensive design, inspection, and permit requirements.

B. Principal Traffic Arteries

Residential and secondary traffic arteries can usually be closed to traffic during the construction of storm sewer crossings. On heavily travelled streets and highways where public safety and convenience are major factors, it may be necessary to use detours, tunnelling, or jacking methods for the crossing.

When required, traffic movements across trenches can be accommodated by temporary decking. Trenches of narrow or medium width can be spanned with prefabricated decks placed on steel or timber mudsills or soldier piles at the edges of the trench. Where the top of the trench is wider than 16–20′ temporary piling for end support, and in some cases center support, may be required.

C. Outfall Structures

Storm sewer outfalls and headwalls may be located above or below surface water levels. When they are partly submerged, it is necessary to provide some form of cofferdam during construction. In shallow water, an earth dike or timber piling may be sufficient to maintain a dry excavation. In deep water, steel sheet piling cofferdams are desirable. Usually a single wall cofferdam with adequate bracing is sufficient, but in excessive depths at the banks of main navigation channels, a double wall may be required. Standard cofferdam design and construction practices should govern.

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XVI. CONSTRUCTION RECORDS

It is generally the responsibility of the contractor to record details of construction as accomplished in the field. These data should be transferred to the engineer for incorporation into a final revision of the contract drawings, so that they may be available for future use.

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Appendix A

PLANNING AND DESIGN EXAMPLES

This appendix includes two examples, both of which are related to planning. These examples were selected to illustrate principles detailed in the Manual, and to demonstrate how they may be practically applied. The first is an example of a plan to control existing and future impacts of stormwater pollution on the water quality of two drinking water reservoirs in Newport News, Virginia. The second describes the planning, design and construction of a multi-purpose flood control facility in Valparaiso, Indiana.

I. RESERVOIR WATER QUALITY PROTECTION—NEWPORT NEWS, VA

A. Introduction

Newport News, a city in southeastern Virginia, faced a problem of water supply reservoir contamination due to nonpoint source pollution stemming from urban development. This example describes a plan to control the existing and future impacts of stormwater pollution on the water quality in the two terminal water supply reservoirs of the Newport News waterworks system, Lee Hall and Harwood's Mill, whose watersheds are 15.8 mi^2 and 9.5 mi^2, respectively (see Figure A.1).

The plan uses "best management practices" (BMPs). There are two basic approaches possible to manage urban nonpoint pollution with structural BMPs:

(a) Onsite Control, which involves the construction of individual BMPs (e.g., wet detention basins, infiltration facilities), designed and constructed by individual developers on each development site.
(b) Regional BMP Master Plan, which involves the strategic location of BMP facilities to simultaneously control nonpoint pollution loadings from multiple development projects. These facilities would be con-

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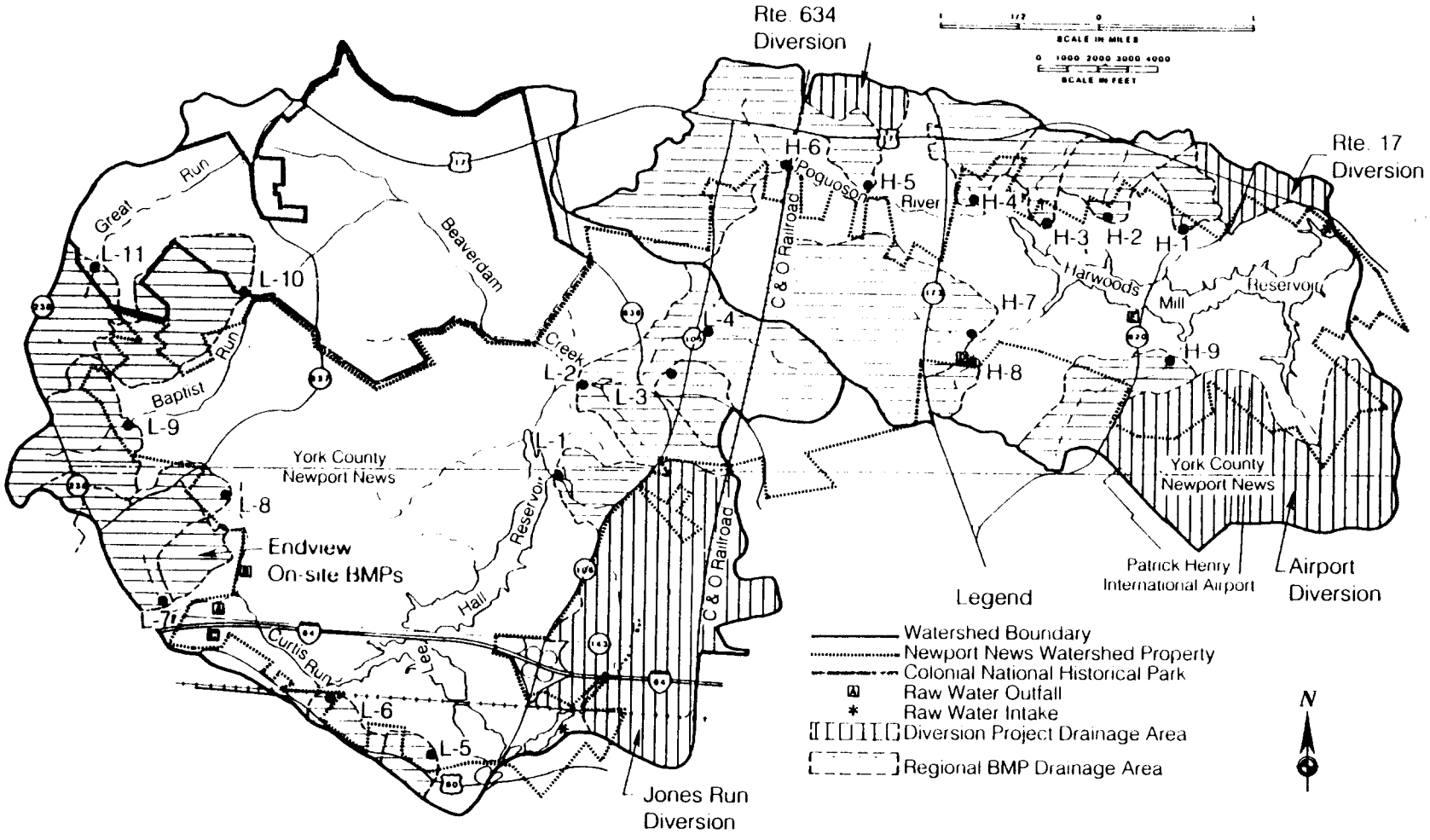

Figure A.1—Regional BMP master plan (ft × 0.304 8 = m, mile × 1.609 = km).

structed and operated by local government. Capital costs would be recovered by pro-rata contributions by the developers, or by other financing mechanisms.

Virginia has operated local stormwater management programs for peak flow since the mid-1970s, and local governments typically elected to use the onsite control approach because it seemed to require little advance planning and appeared relatively easy to administer. Now, however, many local governments in Virginia are giving greater consideration to the regional approach. The main advantages of this approach include:

(a) Reduction in Capital and O&M Costs: Larger facilities have economies of scale, in terms of costs of construction, land acquisition and engineering design. EPA NURP studies indicate that average annual O&M costs for detention basin BMPs are about 3%–5% of base construction costs, and regional facilities should achieve corresponding economies of scale. Moreover, larger facilities can more easily be designed for ease of maintenance.
(b) Increase in Land Development Opportunities: Less total land is needed for large facilities, and the regional approach typically relies on existing floodplain areas for much of the reservoir surface area.
(c) Increased Opportunities for Recreational Use: Regional facilities are larger, and generally can be designed to provide convenient access.
(d) Opportunity to Manage Nonpoint Pollution Impacts from Existing Development: Some existing development may be located within regional BMP watersheds, and nonpoint control will be achieved.
(e) Improved Peak Flow Control: Because they can be strategically located within a watershed, regional facilities can be sited to ensure more reliable peak flow control than randomly-located onsite controls.

Disadvantages of the regional approach include:

(a) Local government must perform advance studies to locate and develop preliminary designs for such facilities.
(b) Local governments must finance, design, and build the regional facilities before most development occurs. Reimbursement by developers will be delayed until building is complete.
(c) Local governments may incur extraordinary maintenance responsibilities if BMPs are perceived as primarily recreational facilities, which merit commensurate water quality protection.

B. Regional BMP Siting Considerations

Figure A.1 also shows the locations of the 24 regional BMP facilities recommended in the study, designated by the prefix "L" and "H" for the Lee Hall and Harwood's Mill watersheds respectively (for four of the highly commercialized/industrialized watersheds, the recommended plan was to divert this runoff to another receiving water). Factors considered in their location were:

(a) Type of BMP (two types were considered): (1) infiltration controls, which divert stormwater runoff into the soil where pollutant removal occurs through such processes as filtration, adsorption, and oxidation by soil microorganisms; and (2) detention controls, which remove pollutants by sedimentation and, in the case of wet detention basins, through physical, chemical, and biological processes which take place in the basin's permanent pool.
(b) Drainage area restrictions: in this case, large enough to serve more than one development, but not so large as to require a permit under the State's Dam Safety program (which would cause undue delays and greater administrative costs).
(c) Land availability (with highest priority given to sites owned by Newport News Waterworks) and Tributary Land Use: Criteria included appropriate drainage area; sufficient amount of proposed and/or existing urban development; design pool elevations that did not impact on upstream structures, roads or other facilities; adequate access for inspection and maintenance.
(d) Ability to control future land use.
(e) Encroachment upon existing urban areas, roads and utilities.
(f) Comprehensive coverage—In addition to the 20 wet detention basin BMPs selected, major diversion projects were recommended for four areas along watershed boundaries to supplement the detention facility network.

C. General Design Criteria

Because this was a conceptual planning study, only general design criteria were specified. At the time of preliminary design, each facility should receive detailed analyses. Some key design parameters include:

(a) Permanent Pool.
  (1) Storage requirements (volume)—Determined in this study on the basis of land use.
  (2) Depth—Shallow enough to minimize thermal stratification and maintain aerobic bottom conditions, but deep enough to prevent light stimulation of rooted aquatics in the open water zone. Mean depth chosen is about 1–3 meters, and maximum depth to prevent stratification of about 5–6 meters.
  (3) Side Slopes—No steeper than 4:1 (for safety), and planted from 2′ below to 1′ above the permanent pool control elevation for ecologic balance and safety.
(b) Earth embankment—Suggested criteria taken from U.S. Soil Conservation Service Standards and Specifications (1981). These include minimum top widths of 6′ (for embankments <10′ high) to 8′ (for embankments between 10′ and 15′ high); combined upstream and downstream side slopes not < 5:1, with neither slope > 2:1; freeboard above the permanent pool of at least 2′ (with a spillway to carry the 100-year runoff); and a foundation cutoff to prevent leakage from the reservoir.
(c) Outlet structures—Riser pipe or standard precast unit.
(d) Maintenance easement—Permanent easement at least 15′ in width around the perimeter of the basin, measured from the maximum elevation of the storage pool.

D. Recommended Regional BMP Facilities Plan

The general characteristics of each of the recommended BMP facilities are shown in Table A.1. Sites L-1 and L-2 are existing facilities that would be dredged and maintained as BMP facilities. In addition, there are four diversions. With respect to the four watershed diversions noted above, the Jones Run diversion would bypass runoff from a 921-acre area to a point downstream of Lee Hall dam, and would include a dry detention basin upstream of the railroad crossing of Jones Run (sized for the 100-year storm). The Route 17 diversion would bypass runoff from a commercial and residential area along Route 17 to a point below Harwood's Mill dam. The Airport diversion would bypass runoff from a 1,130-acre area programmed primarily from industrial development. The Route 634 diversion would bypass runoff from 104 acres of proposed industrial development into the Chisman Creek watershed.

E. Water Quality Benefits

The impacts of the regional BMP plan were studied using several water quality models. Eutrophication impacts were evaluated with the Rast, Jones, and Lee input/output model (1983) and the Jones and Bachman input/output model (1976). Heavy metals impacts were evaluated with the Dillon and Rigler input/output model (1974). The recommended plan is projected to achieve mean concentrations of chlorophyll-a, nutrients and heavy metals which are lower than those associated either with the onsite BMP approach, or with the existing land use (because regional BMPs will capture runoff from existing, as well as new, development, and the released water will be of better quality than that of the current combined runoff from existing development/undeveloped land).

F. Project Priorities

To assist in the development of an implementation program, the 24 projects were prioritized based on the following criteria.

(a) Existing development—Regional BMPs serving currently developed areas get a higher priority than relatively undeveloped areas.
(b) Future land use—Areas proposed for intensive urban development merit a higher priority.
(c) Location in the watershed—Slug loadings from thunderstorms pose a greater potential threat if the discharge point is relatively close to the raw water intake in the reservoir, and BMPs with discharges near intakes are of higher priority than those discharging to reservoir headwaters.
(d) Permitting—Sites requiring a dam safety permit may merit a higher priority because of lead times.
(e) Land acquisition—Sites on private property merit higher priority than sites on public lands, because of development potential.

TABLE A.1. Regional BMPs: Wet Detention Basins.

Site (1)	Drainage Area (acres) (2)	Storage Volume (acre-ft) (3)	Priority (4)
Harwood's Mill			
H-1	86	22.9	B
H-2	161	33.0	B
H-3	107	17.6	B
H-4	163	29.2	A
H-5	110	19.2	A
H-6	564	86.3	A
H-7	626	72.9	D
H-8	67	19.2	C
H-9	228	32.8	B
Lee Hall			
L-1	117	14.4	C
L-2	806	55.0	B
L-3	114	27.3	D
L-4	318	47.2	D
L-5	45	11.3	B
L-6	69	7.9	B
L-7	230	36.0	A
L-8	75	10.9	B
L-9	348	53.3	C
L-10	358	45.5	C
L-11	226	25–40	C

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Assigned priorities for each of the regional BMPs are shown in Table A-1. The Jones Run and Route 17 diversions were assigned priority "A," the Airport diversion was assigned priority "B," and the Route 634 diversion was assigned priority "C."

G. Cost Estimates

Construction costs were estimated using a regression equation derived for the Washington, D.C. region (U.S. Environmental Protection Agency, 1983) that relates construction cost to storage volume, and O&M costs were estimated to be 3%-5% of base construction costs.

H. References

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II. CASE STUDY OF A MULTIPURPOSE FLOOD CONTROL FACILITY

A. Introduction

Successful implementation of a multi-purpose project typically requires a series of complex planning, design, financing, and construction steps involving many public and private sector participants. The complexity of the planning-through-construction process can mean that even technically feasible and economically attractive projects involving storm water detention may never be achieved.

This case study describes the planning, design, financing, and construction of a flood control-recreation facility, together with related public works projects. Although the case study is project-specific, many aspects can be extrapolated to other potential multi-purpose facilities.

B. Recent History of Infrastructure Needs

1. *Description of City and Area*

The flood control-recreation project and related watershed described in this case study are within and near the City of Valparaiso, Indiana. Valparaiso has a population of 22,000 and is economically related to, but not contiguous with, the large Chicago metropolitan area.

Project area climate is characterized by markedly different seasons with corresponding variations in temperature and in precipitation type, amount, and intensity. Surface water related problems such as flooding and erosion fluctuate during the year, but are generally most severe during spring and summer.

2. *Flood Control Needs*

Serious flooding occurred in recent years, most notably in June and July, 1981 and again in July, 1983. In each instance, overland flooding and sewer backup caused widespread damage and disruption. The last rainfall was the most severe. A total of 7.3 inches of rain fell in 27 hours and 6- and 13-hour portions of the event had recurrence intervals in excess of 100 years. Flooding problems were especially serious in the Smith Ditch watershed (see Figure A.2).

In October 1983, the city commissioned the preparation of a comprehensive flood control plan for the Smith Ditch watershed and one other problematic watershed. The planning project was completed in June 1984 (Donohue and Associates 1984).

The following guidelines were developed to provide the basis for and give direction to the planning process, and were used in the development and evaluation of alternatives:

(a) Configure and size facilities to store or convey runoff from the 100-year recurrence interval, 6-hour rainfall occurring under future land use conditions (the 6-hour duration was selected on the basis of sensitivity analyses).
(b) Resolve all flood problems as close to their point of origin as possible and avoid shifting problems from one location in the watershed to another.
(c) Begin to resolve sanitary and combined sewer backup problems by addressing the extensive and serious surface flooding problem.
(d) Favor gravity inflow and outflow for detention/retention facilities.
(e) Give preference to a few large, publicly owned and maintained detention/retention facilities rather than many small privately owned facilities.
(f) Consider the recreational and aesthetic aspects of potential detention/retention facilities.

Two digital computer models, HEC-1 and HEC-2, developed by the U.S. Army Corps of Engineers' Hydrologic Engineering Center and available in the public domain, were used to analyze the system and

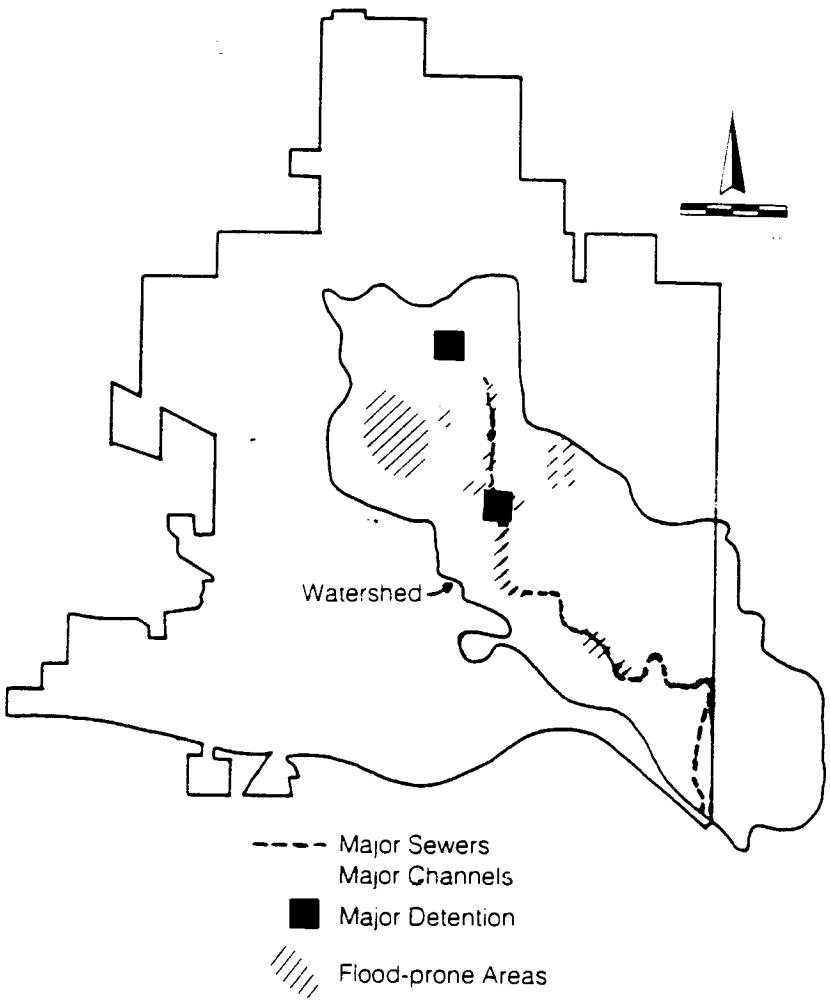

Figure A.2—Smith Ditch watershed (mile × 1.609 = km).

explore alternative solutions. HEC-1 was used to determine the volume and timing of run off from the land surface to watershed swales, channels, storm sewers, and detention/retention facilities, and to route that runoff downstream through the conveyance and storage system thereby producing a series of discharges at predetermined locations. HEC-2 was used to calculate flood stages for selected reaches in the open channel system. Figure A.3 shows input to, output from, and inter-relationships between the two computer models.

The relationship between important components of the watershed system and the computer programs used to simulate them are illustrated in Figure A.4. All of the land surface, most of the conveyance system, and all detention/retention facilities were modeled with HEC-1. HEC-2 was applied to those channel reaches requiring detailed hydraulic analyses. The total of 19 simulations that were conducted included existing and future land use, a historic rainfall event, and rainfall events over a range of recurrence intervals.

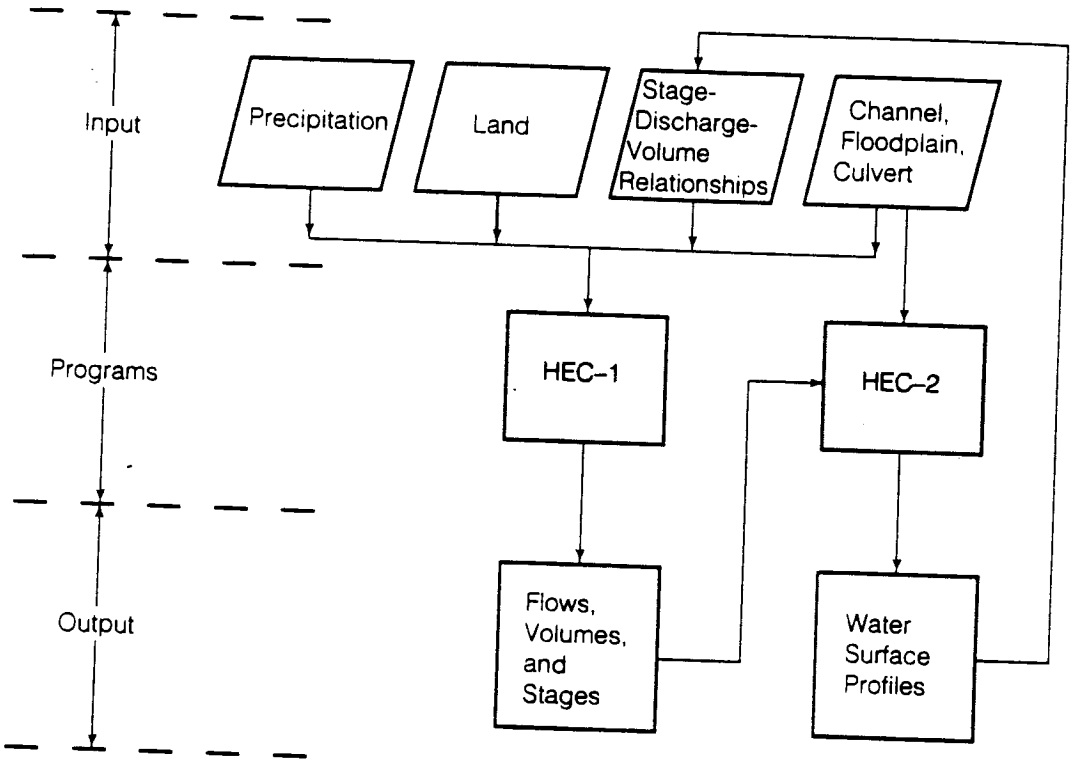

Figure A.3—Hydrologic-hydraulic model composed of two HEC programs.

The diagnosis of the hydrologic-hydraulic system using simulation indicated that existing detention facilities were undersized, and major channels and conduits through built-up areas had insufficient capacity. Future urbanization would aggravate flooding in scattered locations throughout the watershed.

Various alternative structural flood control facilities were considered in the planning process. For example, the technical and economic feasibility of diverting flood water from the upper portion of the Smith Ditch watershed to the contiguous watershed on the west was screened. This alternative was rejected because of the high cost, as well as the fact that it would be in conflict with one of the planning guidelines established early in the project, which called for resolving flooding problems as near as possible to the point of origin and not moving them from one area to another.

Recommended major improvements, as illustrated in Figure A.5, include the construction of two new detention facilities (one of which is the subject of this paper), modification of two existing detention facilities and extensive channel cleaning and maintenance. The plan also recommended that the city develop and implement a comprehensive program for operation and maintenance of sewers and channels; that owners or renters of residential or commercial property consider purchasing flood insurance; that the city review, codify, and expand its

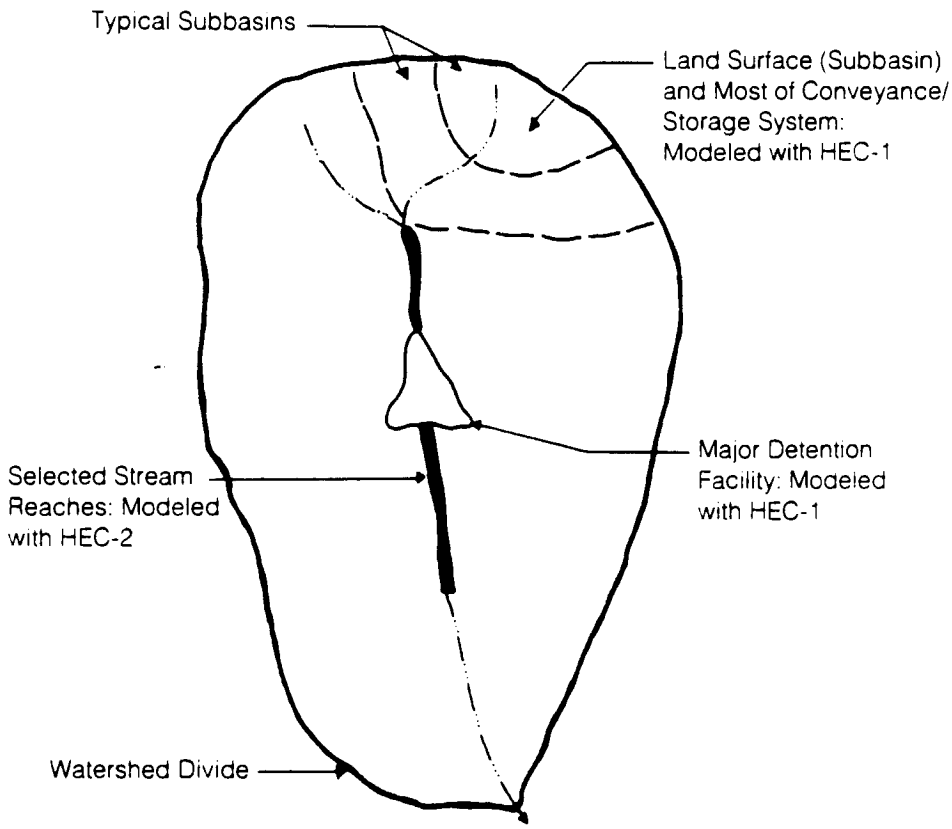

Figure A.4—Watershed components and portions of the HEC-1 and HEC-2 model used to simulate them.

storm water management regulations; and that the city, possibly in cooperation with the surrounding county, develop a current comprehensive stormwater management plan for all watersheds in the Valparaiso area.

Since issuance of the plan, the recommended channel cleaning has been completed, the two existing detention facilities have been enlarged as recommended, and the larger of the two recommended detention facilities has been designed and constructed. Partly as a result of the plan, the city has also taken a lead role in seeking state legislation to enable an Indiana municipality to establish a stormwater management utility. The legislation was adopted and became effective September 1, 1987 (Indiana 1987). Other structural and non-structural recommendations are under consideration by the city.

As shown in Figure A.6, most of that portion of the Smith Ditch watershed subject to flooding has been extensively urbanized. Residential and commercial land uses cover most of the upper two-thirds of the watershed. Undeveloped lands consist mostly of privately held property intended for development.

At the time of the planning project, the fairgrounds site, which is located approximately two-thirds up the Smith Ditch watershed, was

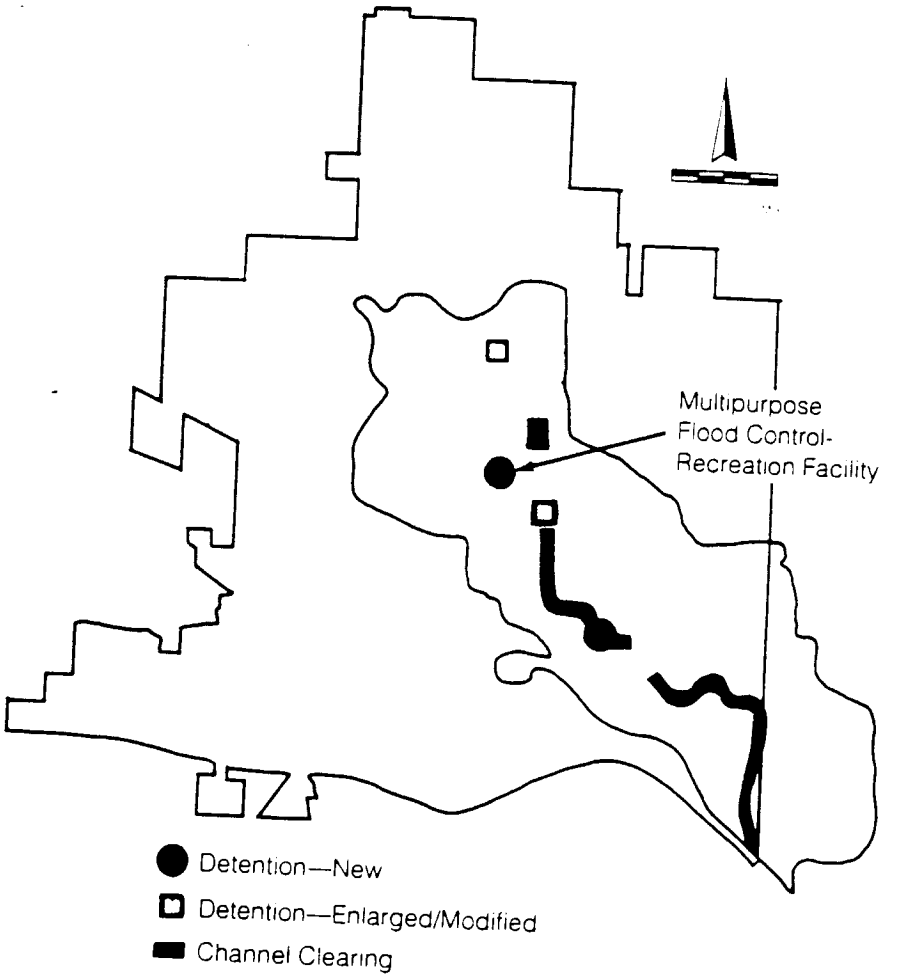

Figure A.5—Recommended major improvements in Smith Ditch watershed (mile × 1.609 = km).

one of the few undeveloped publicly-owned parcels in the watershed. The site, which was then owned by the Board of County Commissioners and operated by the Porter County Fair Board, was last used as a fair site in the summer of 1984. A new site outside of, but close to, the city was being developed and would be ready for use in the summer of 1985. Therefore, at the time of the flood control planning project, the fairground site was potentially available for other uses, including storm water control.

Figure A.7 illustrates the initial concept for the single purpose, off-channel fairgrounds detention facility. As initially envisioned and, from a flood control perspective, as ultimately constructed, the fairgrounds detention facility consists of four major components. The first component is the diversion structure and sewer to carry flow from Smith Ditch at McCord Road into the fairgrounds. The second component is the 90 acre-foot detention facility initially envisioned as occupying about eleven acres of the 27-acre site. The third component is a system of sewers generally flowing from the west and designed to carry storm water

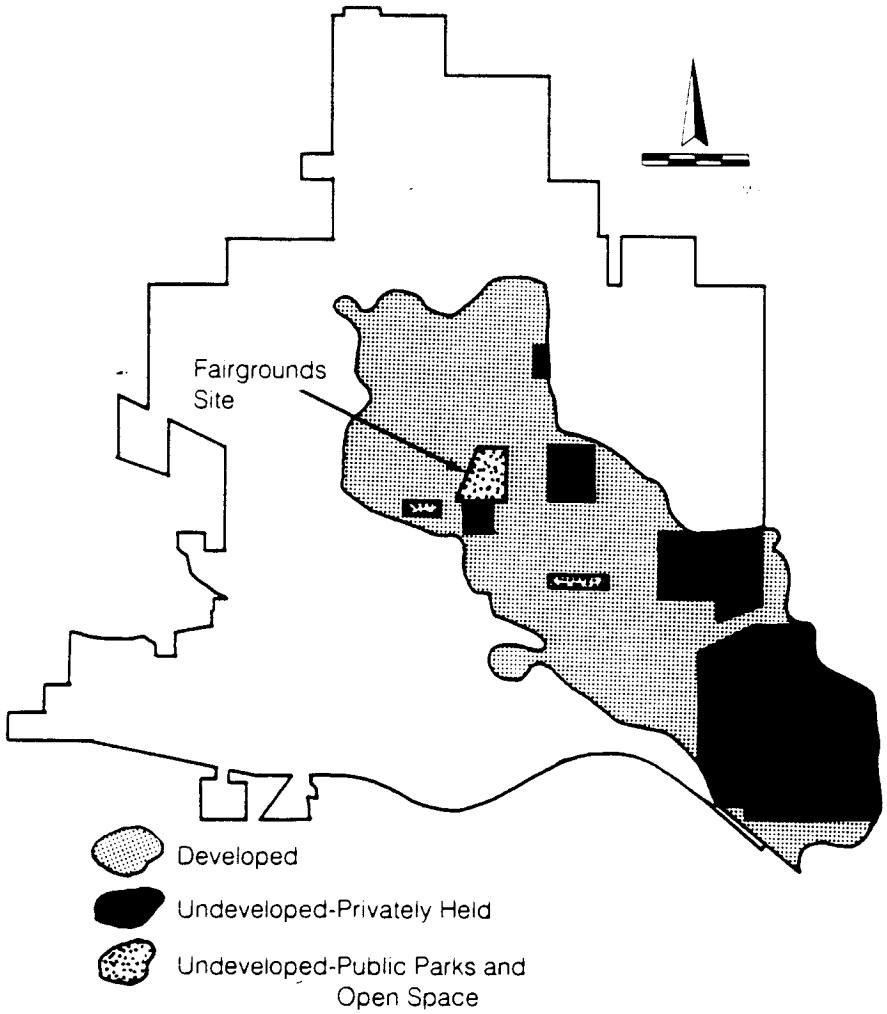

Figure A.6—Land use in Smith Ditch watershed—1984 (mile × 1.609 = km).

from a currently combined sewer area into the detention facility. The fourth and final component is an outlet control, which would slowly release temporarily stored water back into Smith Ditch and would also control flow from the west. Inflow to and outflow from the fairgrounds detention facility would be by gravity, and pumping or automatic controls would not be required.

With respect to flood control, the fairgrounds detention facility will mitigate surface flooding in the residential area immediately to the east along Smith Ditch, reduce surface and basement flooding in the residential and scattered commercial areas to the west, and relieve flooding along Smith Ditch downstream of the facility.

3. *Recreation Needs*

Approximately coincident with the growing concern with flooding problems, the potential recreation needs of the city were receiving

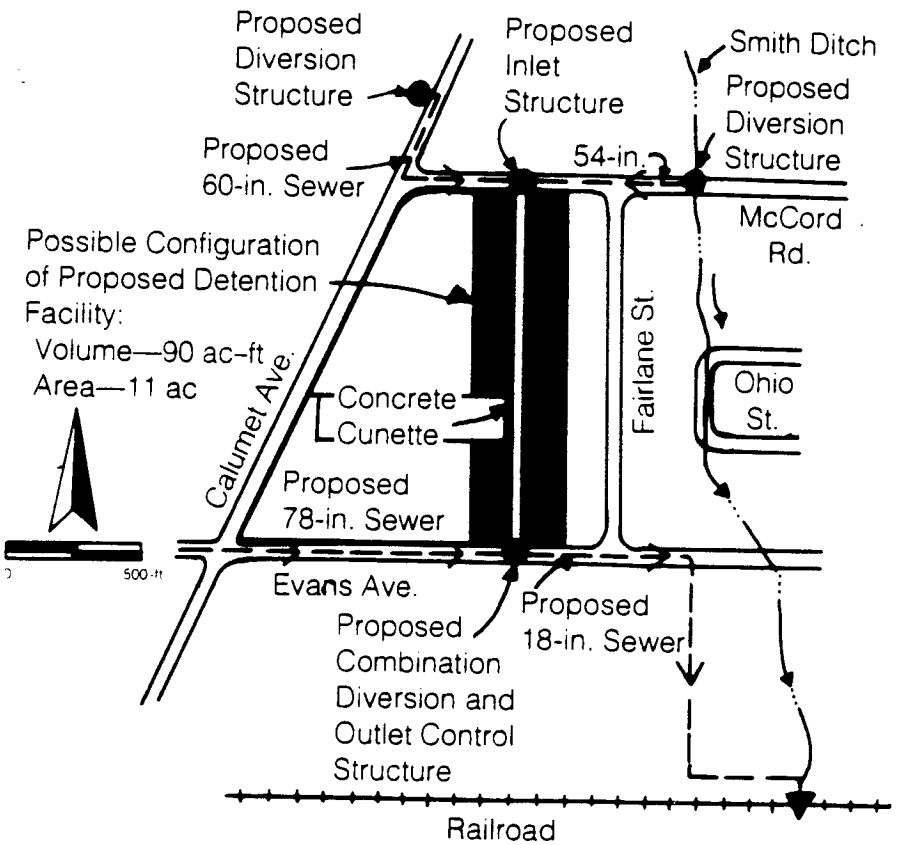

Figure A.7—Initial concept for off-channel detention (ac × 0.404 7 = ha, ac-ft × 12.33 = m^3, ft × 0.304 8 = m, and in. × 25.4 = mm).

increased attention. Population growth and expanding interest in year-round outdoor recreation were placing increased pressure on existing facilities and, at the same time, reducing the availability of open space for development of new recreation facilities. Furthermore, eligibility for federal and state recreation funds required an updated recreation master plan. Accordingly, the Park and Recreation Board commissioned the preparation of a park master plan. The park planning effort, which was carried out from February through August 1984 (Earth Plan Associates 1984), was approximately coincident with the flood control planning effort.

Numerous guidelines were established to provide overall direction to the recreation master planning effort. In retrospect, some of these influenced the eventual decision to develop the fairgrounds site as a multi-purpose flood control/recreation facility. Examples of these guidelines were acquiring land for public park purposes in areas of the city currently void of recreation facilities, establishing cooperative agreements with private and public organizations for joint use of properties, and providing recreation programs for all age groups.

Early in the recreation planning program the fairgrounds site was recognized as offering great potential for a large central city park facility.

Because the park planning project was conducted parallel to and with knowledge of the flood control planning project, the park master plan recommended that a major recreation facility be developed at the fairground site and that it be configured to permit joint use for flood control.

The coincident occurrence of the flood control planning program and the recreation planning program was a catalyst for development of a multi-purpose project. However, the success of the project depended upon the willingness of public officials responsible for flood control and for recreation (and their respective consultants) to work together.

The initial concept for a flood control/recreation development of the fairground site is illustrated in Figure A.8. Recreation facilities included four softball diamonds, three soccer fields, a general purpose athletic field using the existing grandstand for spectator viewing, picnic and playground areas, an ice skating rink, a circumferential jogging/walking/biking trail, parking areas and public use and service buildings. The initial concept envisioned excavating the approximate eastern one-third to one-half of the site to a uniform level to provide the necessary 90 acre-feet of flood control storage.

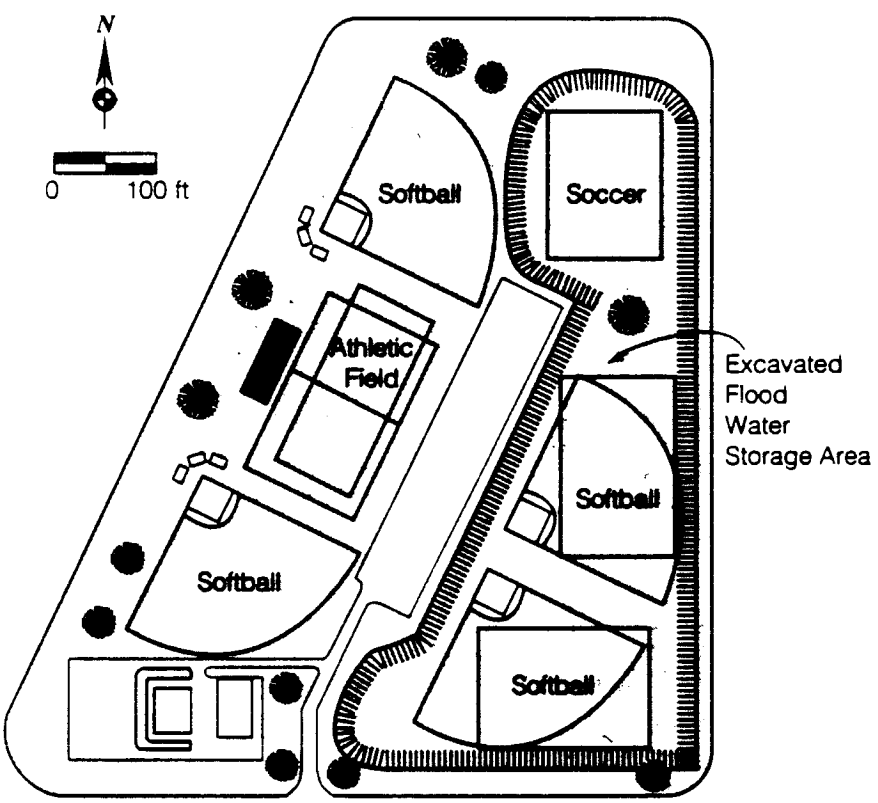

Figure A.8—Initial flood control—recreation plan (ft × 0.304 8 = m).

4. *Highway Needs*

Beginning in the mid-1940's, city, county, regional, state, and federal officials began discussing ways to route north-south highway traffic around the City of Valparaiso. The heavy volume of through traffic (trucks and automobiles having no origin or destination in the city) was the principal concern.

These efforts culminated in 1979 with the completion of a final design study report (Indiana State Highway Commission 1979). A new ten-mile long north-south section of State Road Highway 49 would be constructed. The four-lane limited-access highway would include a bypass along the eastern edge of Valparaiso. Because of a relatively large number of grade separations, extensive fill (several million yards) would be required for the Valparaiso segment.

The first construction contract was let in May 1983 for portions of the bypass north of Valparaiso. Construction on the segment immediately east of Valparaiso began in April 1985.

C. Designing the Multi-Purpose Project

The decision by the City of Valparaiso and the City Park and Recreation Department to design a multi-purpose flood control/recreation facility at the fairgrounds site was made in the summer of 1985. The two consulting firms that originally prepared the flood control and recreation plans were retained to provide design services for the flood control and recreation facilities.

A refined, integrated design was developed incorporating flood control requirements as illustrated in Figure A.7, and recreation requirements as shown in Figure A.8 and Table A.2. The project team, consisting of city engineering and city park personnel, flood control engineers, and park and recreation designers considered and resolved many factors during design such as:

(a) Flattening of side slopes on all excavated areas for ease of maintenance and for aesthetic quality.
(b) Grading of all recreation field surfaces to provide positive and rapid surface and subsurface drainage.
(c) Raising the minimum outlet elevation of the detention facility based on a field survey of local storm sewers specially conducted for the design.
(d) Elimination of the old, unsightly fairground bleachers and rearrangement of recreation facilities, particularly the four softball fields, to provide improved function and less costly concession, restroom, and other service facilities.
(e) Saving of most of the large trees along the east side and in the northeast corner of the site.
(f) Using terraces—relatively flat areas at slightly different elevations—to add topographic variety and interest, in plan and in section, to an otherwise flat or severely excavated site and to decrease the frequency of flooding of some sports facilities.

TABLE A.2. Recreation Facilities

1. *Garden Area*—Located at the southeast corner of site, this area will include trees, shrubs, floral displays, bench seating, lawn development, split rail fence, and entrance sign.
2. *Pavilion*—Renovate existing structure as a landmark to the Old Fairgrounds and as a visual focal point for park. Includes 5000 square feet of seasonal space that can be used for events such as family reunions, wedding receptions, square dances, art shows, and teen dances. Includes restroom and kitchen facilities.
3. *Concession/Restroom/Supervisory Building*—Construct a centrally located building that will accommodate the entire park. Because this central location will also be a visual focal point it is designed to complement the renovated Pavilion and will include extensive landscaping.
4. *Maintenance Building*—Dismantle the existing metal building and use materials at another location. A new maintenance facility will be constructed on the northeast corner and will be designed to complement on-site architecture and be landscaped to blend into surroundings.
5. *Softball Fields*—Provide four fields, two lighted, to accommodate primarily men's, women's and co-ed softball with capability to serve youth baseball/softball.
6. *Soccer Fields/Special Events Field*—Provide one class A field that will serve youth, school and adult soccer and football needs as well as serve as a staging area for a variety of special events. One class B soccer field will be used for special events when available.
7. *Walk/Jog Path*—Consists of one mile of surfaced path for use in all seasons.
8. *Picnic Area*—Includes a wooded area with picnic tables and playground equipment.
9. *Landscaping*—Provide extensive landscaping along the site perimeter with trees, shrubs and flowers.
10. *Parking*—Construct three major paved parking lots to accommodate over 250 cars.
11. *Maintenance - Equipment*—Acquire mowers, aerifier, liners, seeder and small truck to serve the facility.

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The design process, including consideration of the preceding factors, led to the final design illustrated in Figure A.9. A set of terraces cover the site, and the originally flat site now has a total relief of 13.0 feet with the lowest area occurring at the outlet in the southeast corner.

Least used portions of the facility, such as one of the overflow soccer areas, are on the lowest levels. In contrast, frequently used areas, such as lighted softball fields, are on the highest terraces. Frequency and duration of storm water storage for all areas of the site are presented in Figure A.10 (assuming stormwater loads corresponding to ultimate watershed development). As indicated in Figure A.10, the southeast or lowest corner of the facility will be inundated at least once per year for a duration of about two days. In contrast, the softball diamonds will be inundated for a fraction of a day once every ten or more years depending on their specific elevation.

Very close attention was given to the hydraulic function of the facility. The diversion structure on Smith Ditch will divert all flows above 60

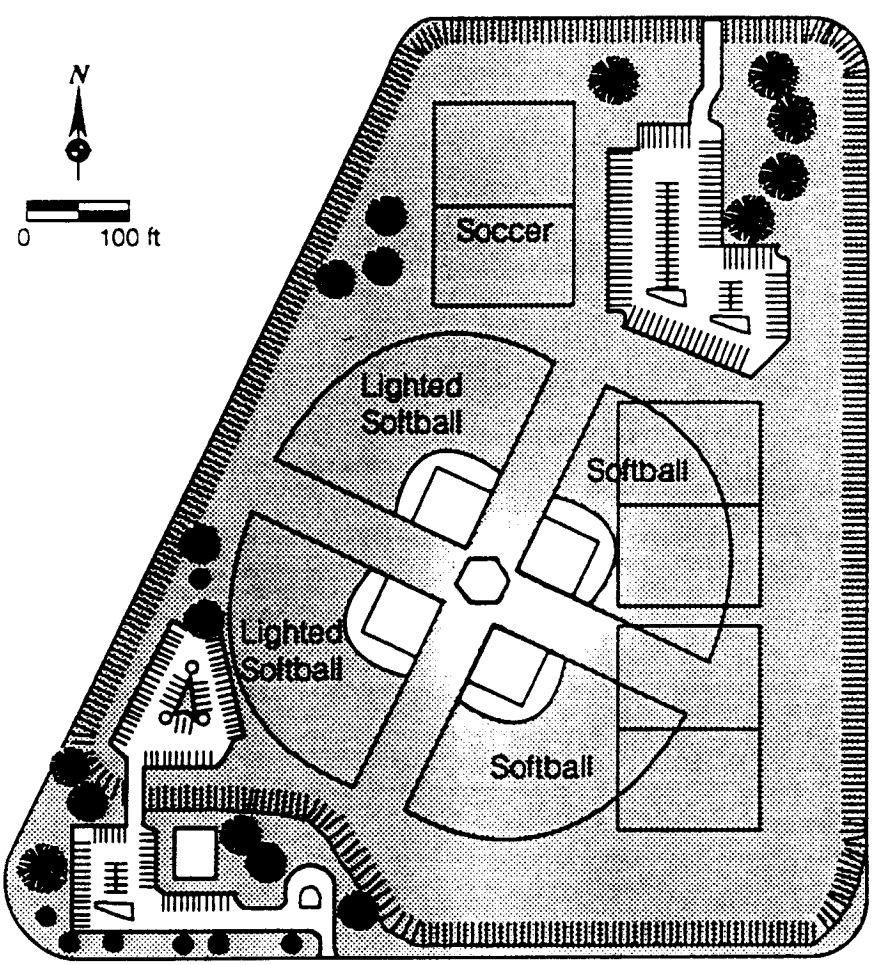

Figure A.9—Final flood control—recreation design (ft × 0.304 8 = m).

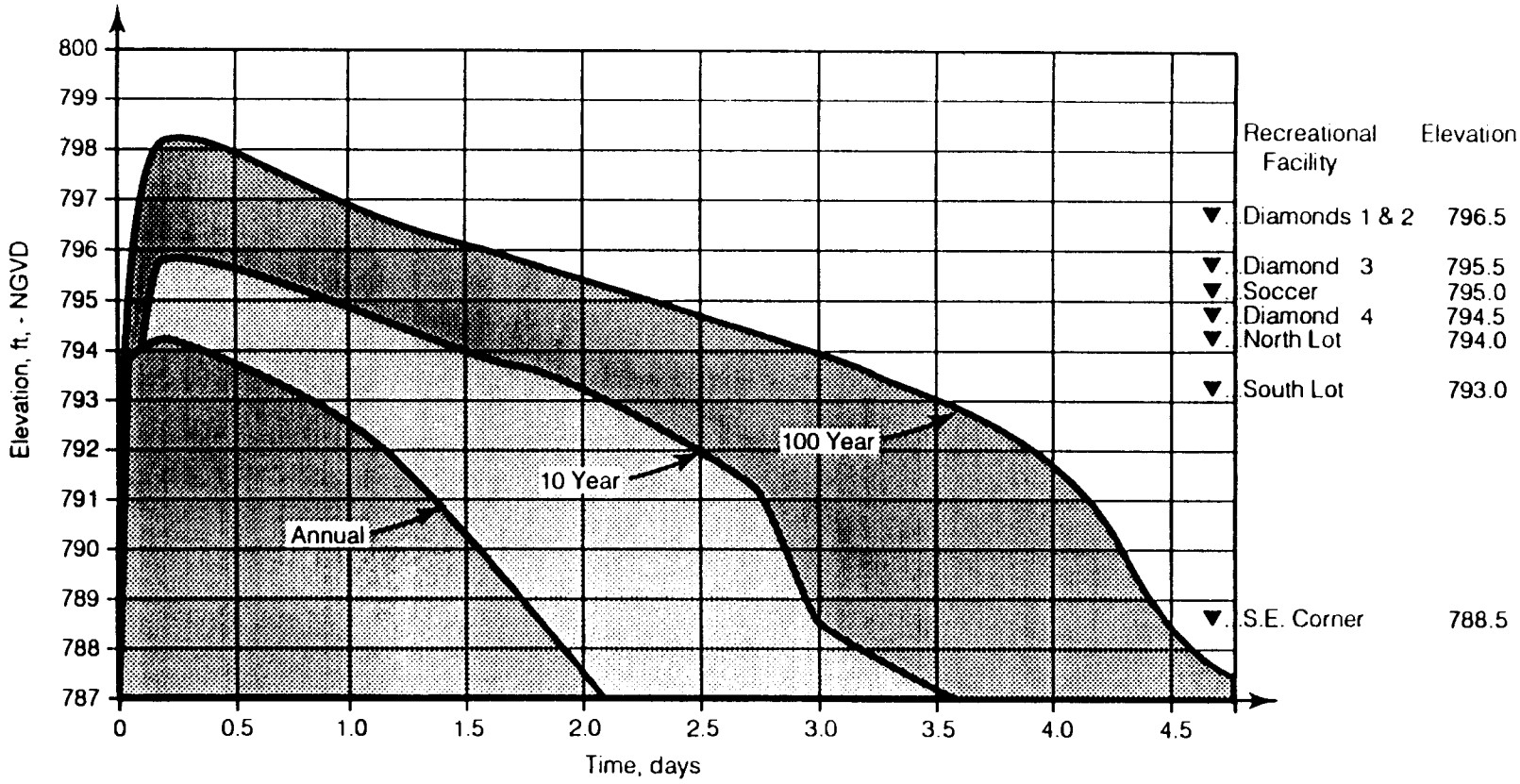

Figure A.10—Frequency and duration of stormwater storage (ft × 0.304 8 = m).

cfs into the detention facility. Flows of less than six cfs that enter the facility from the north will be carried to the outlet control structure in an 18-inch diameter southerly flowing sewer unobtrusively located along the eastern edge of the facility. Flows below six cfs from the west along Evans Avenue will bypass the facility and go directly to Smith Ditch. Excess flows from the west will be temporarily diverted into storage.

An extensive network of 4–18 inch diameter perforated and corrugated polyethylene pipe under drains and sewers provides subsurface drainage of all recreation areas. All recreation surfaces have a surface slope of at least 1% to encourage rapid drainage after rainfall or flooding. Because of the careful attention given to grading and subsurface drainage, even the lower recreation facilities on this site will probably be available for use more often than other single-purpose facilities scattered around the city.

The final design provides numerous and varied active and passive recreation opportunities. The design also includes on-site provisions for service and maintenance facilities and equipment. The value of the constructed project was about $2.4 million.

D. Financing the Multi-Purpose Project

A major public works project like this is typically financed through sale of bonds. Although a bond issue was used to finance much of the cost of the facility, other innovative means of finance were also used as summarized in Figure A.11 and discussed in the following sections.

1. Land Acquisition

The City and the Park and Recreation Department jointly committed to proceed with the multi-purpose project in summer 1985. The city obtained an appraisal of the 27-acre fairgrounds site and began to negotiate with the County for site acquisition. The city purchased the site in August 1985 for $300,000 to be paid in amounts of $50,000 per year at zero interest. As a part of the negotiations, the city agreed to give the County first right of refusal on a City-owned parking lot close to the County court house.

The city and the Valparaiso Community Schools then negotiated an agreement whereby the school system would give the city 11.3 acres of land valued at $254,000. This land had been held by the Valparaiso Community Schools to meet anticipated future education needs. In return for the land, the city agreed to provide certain recreation facilities at the new fairgrounds facility or elsewhere for use by the school system. The effect of this exchange was to offset most of the cost incurred by the city in purchasing the fairgrounds inasmuch as the City could sell the land acquired from Valparaiso Community Schools as a means of recovering the fairgrounds purchase price or other costs incurred in providing the additional recreation services.

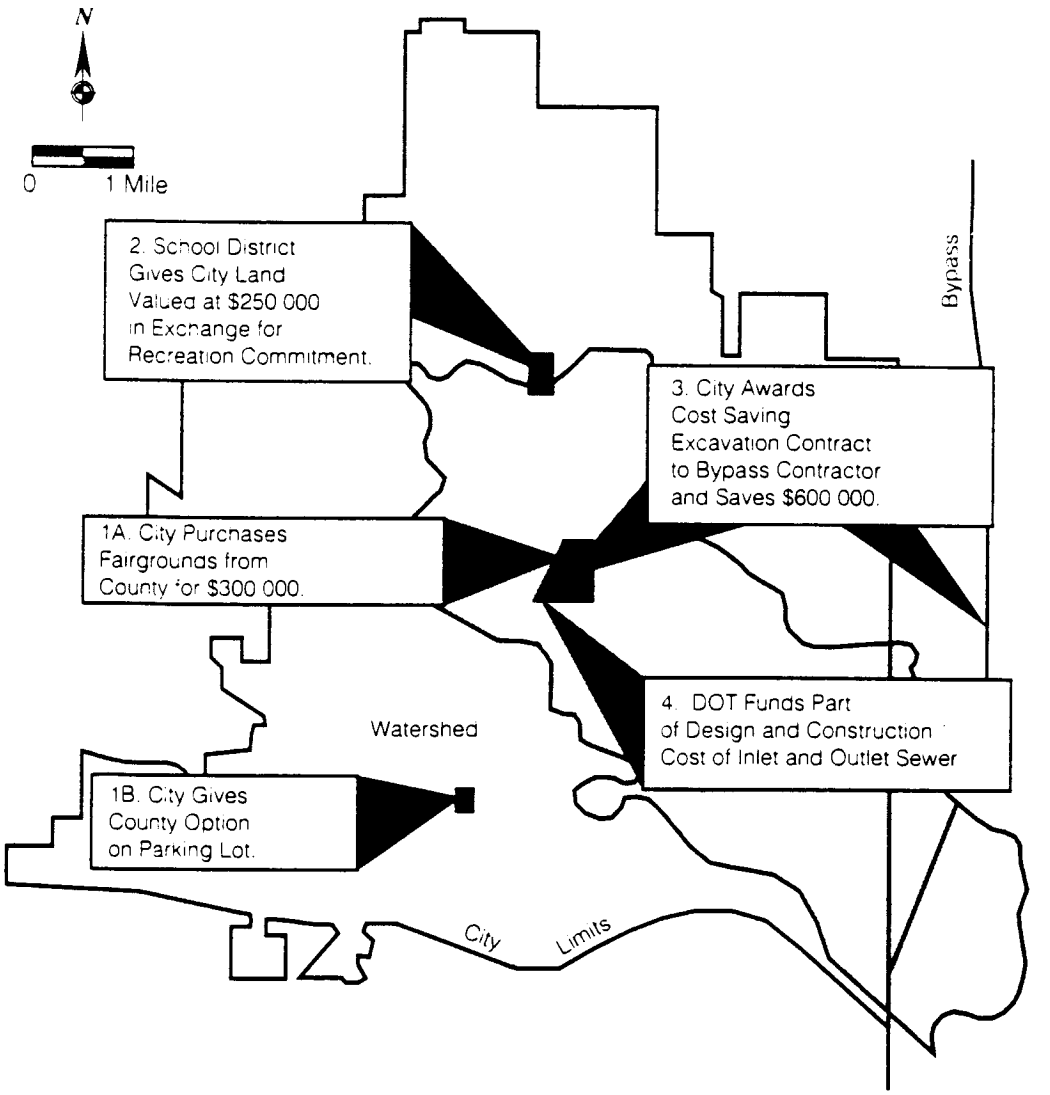

Figure A.11—Financing.

2. *Excavation*

As noted earlier, the construction of the State Road 49 bypass in the vicinity of Valparaiso, which was under way in mid-1985, created a strong demand for fill material. The fill areas were 1.6 or more miles away from the fairground site suggesting that a long haul would cause high excavation costs. However, the city began to investigate the possibility of providing fill for the highway project because projected site excavation costs were a large part—about one third—of the total cost of the project.

As a result of exploring the possibility of tying fairground site excavation needs to bypass fill needs, the city advertised the availability

of up to 210,000 cubic yards of material at the fairgrounds site and received six bids. Listed in decreasing order, the per cubic yard bid prices were $5.05, $5.00, $3.85, $3.29, $2.60, and $1.02. The lowest bid was submitted by a contractor involved in the bypass highway project. The excavation contract was awarded to the low bid contractor who eventually removed 180,500 cubic yards from the site for $1.02 per cubic yard ($184,100) and hauled it to the highway project to meet his contractual obligations with the State Department of Highways. Because of the opportunity to begin excavating in August 1985 at considerable cost savings, the design of the excavation portion of the site began immediately and proceeded quickly.

Assuming that the original estimate of $4.20 per cubic yard for excavation, in 1985 costs, would have prevailed in absence of the tie-in to the highway project, the city saved $574,000 by moving quickly and integrating the fairgrounds construction with the bypass construction. In exchange for a time extension, the excavate and haul contractor subsequently agreed to provide additional services at no additional cost to the city. These savings included stripping and stockpiling topsoil at the fairgrounds site, rough grading, replacing topsoil on the steeper side slopes, seeding and mulching the slopes, and demolishing and removing the concrete grandstand. These supplemental services represented an additional savings of at least $20,000 to the city.

3. *Tie-In to Street Improvement Project*

Prior to the planning of the fairgrounds project and completely unrelated to it, the city, state, and federal government had been designing and preparing financing for improvements to a portion of Calumet Avenue which borders the fairgrounds site to the west. Included were necessary drainage facilities leading to a large sewer to carry stormwater from Calumet Avenue to Smith Ditch.

Construction of the storm sewer to Smith Ditch would have negated some of the flood control benefits being provided by the fairgrounds project and would have resulted in unnecessarily expensive sewer construction from Calumet Avenue to Smith Ditch. Furthermore, the fairgrounds facility was sized to accommodate future development runoff from areas west of the site.

Accordingly, the drainage component of the Calumet Avenue highway improvement project was altered to direct runoff into the fairgrounds facility. This reduced the cost of the street improvement project.

Federal and state transportation funds are being used to pay for some of the design and construction costs associated with the outlet control. City design and construction costs were reduced $90,000 as a result of federal and state participation.

4. *Other Savings*

The city used its new Project Management Office (PMO) system for this project, thus saving the usual additional fee for a general contractor. City personnel performed full time inspection and had full time control. The PMO approach yielded further cost savings.

E. Constructing the Multi-Purpose Project

Excavation began abruptly in August 1985 as a result of the demand for fill at the nearby highway bypass project. Flood control and recreation facilities were both first used in 1990.

The city staff is designing a separate storm sewer for the now partially combined sewer residential area west of the fairgrounds site. The city is also designing major storm sewers to convey stormwater from the residential area to the west directly into the fairgrounds facility.

F. Building Community Support for the Multi-Purpose Project

Numerous public information events and activities, beginning early in the planning process, were conducted for the flood control and recreation aspects of the project. Early activities included a neighborhood meeting and field reconnaissance of flood prone areas by project engineers during which numerous personal contacts were made with citizens. Other public information events and activities included presentations at City Council meetings, presentations at Park Board meetings and newspaper articles.

Interest in flood control works tends to rise and fall with the flood waters. If the fairgrounds project had been for the single purpose of flood control, community interest may not have been continuously sustained, in spite of the public information efforts, at a high enough level to see the project through to implementation. The recreation features of the project helped to broaden and sustain community interest. Project implementation benefited from the sense of urgency created by both the availability of the fairgrounds site (because of the imminent relocation of the annual county fair) and by the immediate need for major amounts of fill at the bypass highway project.

G. Summary and Conclusions

Multi-purpose public works projects often "look good on paper" because they can be shown to be technically feasible and economically attractive. However, implementation of such projects is difficult because of the planning, design, financing, and construction steps required, and the need to involve many participants.

The case study presented in this paper suggests ways to deal with the complexities inherent in multi-purpose public works projects. Key factors to the success of the fairgrounds flood control-recreation project

were sound technical design utilizing watershed computer simulation, a continuous public information effort, imaginative financing and trading, and quickly seizing unexpected, mutually beneficial opportunities.

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WATER ENVIRONMENT TECHNOLOGY

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Model predicts metals in stormwater

Wetlands treat runoff and effluent

The key to successful stormwater education

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Vol. 8, No. 7
July 1993

WATER ENVIRONMENT & TECHNOLOGY

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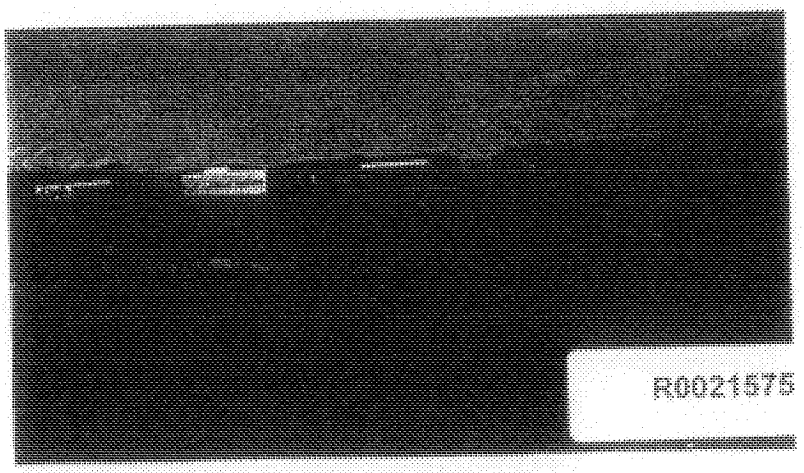
The new stormwater management program will eliminate a major source of surface water quality degradation.



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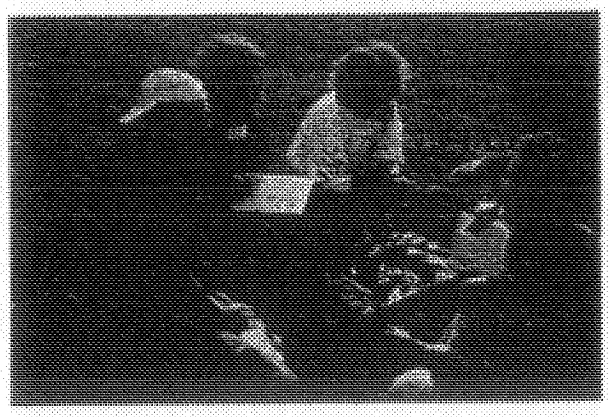

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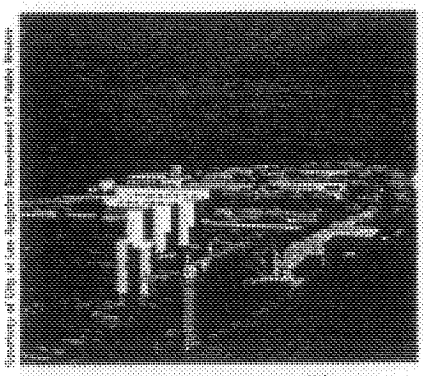

Photo courtesy of City of Los Angeles Department of Public Works

The Hyperion Treatment Plant in Los Angeles, Calif., received an Excellence in Environmental Engineering award in the operations-maintenance category.

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Water Environment Federation
Formerly Water Pollution Control Federation

Water Environment & Technology (ISSN 1044-9469) is published 13 times per year, monthly, with a special issue in April. ©1993, by the WATER ENVIRONMENT FEDERATION. Correspondence should be sent to Water Environment Federation executive and editorial offices: 601 Wythe St., Alexandria, VA 22314-1994.

Subscriptions: (703) 684-2400. Subscription included with individual FEDERATION membership of $60/year plus member association dues. Rates for nonmembers: $144 in the U.S., $187 elsewhere. Regular single issues: $12 members, $14 nonmembers. Add $10 in single issue cost outside the U.S. Single issue prices include postage and handling. Orders under $50 and orders outside the U.S. must be prepaid.

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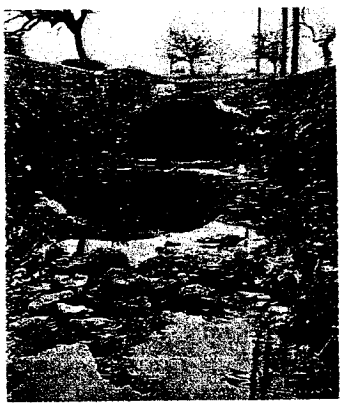
PRELUDE

Stormy Weather



Aldo Leopold described stormwater romantically in his *Sand County Almanac* in the early part of this century. He wrote that the spring flood brings, "more than high adventure; it brings likewise an unpredictable miscellany of floatable objects pilfered from upriver farms. An old board stranded on our meadow...has its own individual history, always unknown, but always to some degree guessable."

Unfortunately, stormwater now brings more than, "an anthology of human strivings in upriver farms and forests." With increased urbanization, our surface water currently receives nutrients; sediments; inorganic metals, such as lead, copper, zinc, and chromium; and organic chemicals, such as pesticides and solvents.

Early efforts to *control* stormwater runoff were aimed at reducing flood damage. Now, stormwater drainage and discharge facilities that channel runoff into storm sewers are no longer sufficient by themselves. Stormwater *management*, including effluent standards, is required.

According to an article by former EPA Administrator William K. Reilly, "Beneath Los Angeles, more than 1000 miles of storm drains collect runoff from city streets, dumping it in coastal bays. The Natural Resources Defense Council estimates that in 1989 eight inches of rain washed 150,000 pounds of lead, 500,000 pounds of zinc, and 11,000 pounds of cadmium into Santa Monica Bay alone."

EPA has chosen to address the urban and industrial stormwater runoff problem through a traditional permitting program. EPA now requires stormwater permits under its special National Pollutant Discharge Elimination System (NPDES) program (*40 CFR* Part 122.26). In addition, states are allowed to regulate stormwater discharges under federal guidelines.

Stormwater permit applications need to include information such as: a map showing facility topography, drainage and discharge structure locations, paved areas, and outdoor storage areas; an estimate of impervious surfaces; a description of the materials that have been exposed to stormwater; a certification that stormwater discharges and outfalls have been tested; and analytical data on samples collected during storm events, NPDES permitted pollutants, and flow measurements.

Stormwater is the focus of this issue of *Water Environment & Technology*. The feature topics include regulating stormwater metal toxicity, using wetlands to treat stormwater, promoting stormwater education, and complying with the NPDES stormwater permitting process.

The new stormwater management program will eliminate a major source of surface water quality degradation. So, let it "...keep on rainin' all the time."

Elizabeth F. Wilkins
Editor

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EPA Gives Views on Clean Water Reauthorization

The Clinton Administration's views on reauthorization of the Clean Water Act were presented by EPA administrator Carol Browner on May 4, 1993, before the House Subcommittee on Water Resources and Environment. Funding, watershed protection, toxics, pollution prevention, contaminated runoff, and enforcement were the key issues discussed as priorities in the act's reauthorization. Although no specific amendments were addressed, Browner called for adequate funding for stormwater, combined sewer overflows, and agricultural runoff initiatives. While the administration will not develop a comprehensive reauthorization proposal, it will work with Congress to develop a bill. A list of priority issues and suggestions for addressing them may soon be released by EPA.

Dredged Material Dumping Site Proposed

A new dredged material disposal site off the coast of Norfolk, Va., has been proposed by EPA (FR 27976-27980, May 12). The material is dredged from entrance channels of the lower Chesapeake Bay, which provide access to the Hampton Roads and Baltimore ports. Because these two ports experience heavy marine traffic, the channels require frequent dredging; the old disposal site has reached capacity. The dredged material will have to meet current ocean dumping standards before it can be buried at the new site. To comment or receive more information, contact *William C. Muir of the EPA Environmental Assessment Branch at (215) 597-2541.*

Criteria Released for Evaluating Toxics

EPA has issued criteria for determining if amounts of toxic substance production, release, and human exposure are enough to warrant testing (FR 28736-28749, May 14). These criteria will help the agency implement its authority to permit, ban, or require testing of chemicals under the Toxic Substances Control Act. The criteria will continue to be refined as the chemical testing process becomes more advanced. For a list of these criteria, contact *Susan B. Hazen of the Office of Pollution Prevention and Toxics at (202) 554-0551.*

EPA May Receive Budget Cuts for 1994

The Clinton Administration has proposed to cut EPA funding by $600 million for 1994. Of the $6.4 billion designated for the EPA budget, $4.4 billion will go to "core" programs, including the Superfund program (receiving $1.496 billion—an $89 million cut) and the water quality program (receiving $464 million—a $20 million cut). The air quality program, however, may receive a $33-million increase in funding, and the multi-media program may receive an additional $68 million.

EPA Proposes Removal of Substances from TSCA Inventory

EPA has proposed to remove 39 substances from the Toxic Substances Control Act (TSCA) inventory, which is a compilation of chemical substances that have been manufactured, imported, or processed in the U.S. for commercial purposes since January 1, 1975 (FR 19251-19252, April 13). Most of these substances were incorrectly reported to the inventory by individuals who then acknowledged that an error had been made and requested that the chemical be removed from the list. The chemicals either were not manufactured for purposes specified by TSCA or had been described more accurately through improved technology since the listing. In some cases, they may have been added to the list through typographical error. For a list of the chemicals, contact *Susan B. Hazen of the Office of Pollution Prevention and Toxics at (202) 554-0551.*

Wood Protection Wastes May Be Listed as Hazardous

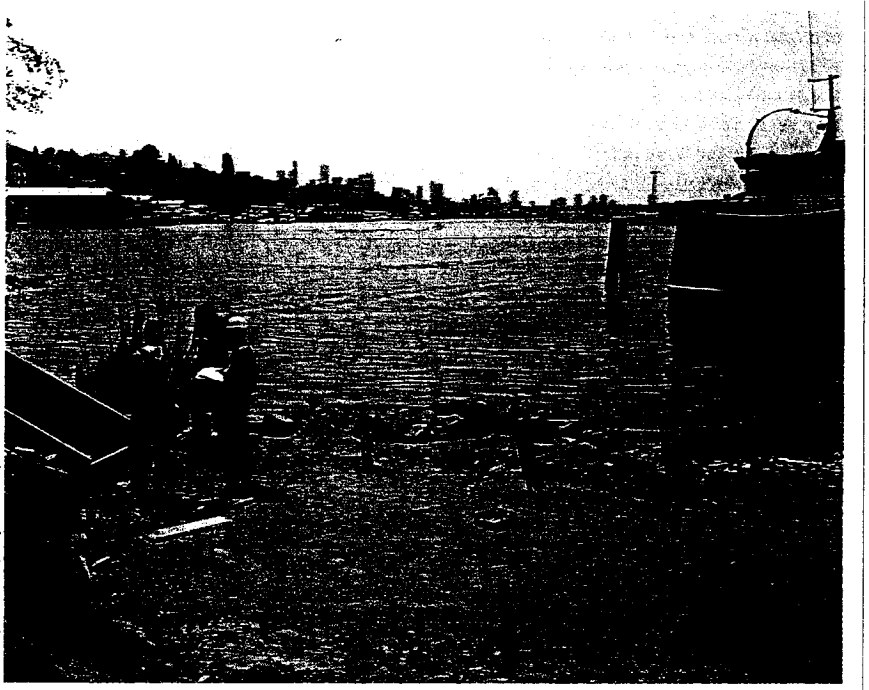
Certain wastes from the wood surface protection industry may be listed as hazardous under an EPA proposal to amend Resource Conservation and Recovery Act regulations (FR 25706-25737, April 27). The wastes may be listed as hazardous if a pentachlorophenate concentration greater than 100 ppb is present during in-process formulation. The proposal suggests testing, analysis, recordkeeping, and management standards requirements for wood surface protection plants. Amendment of the Superfund list of hazardous substances to reflect the new listing of substances contained in the wood protection waste would also be required under the proposed rule. For more information, contact the *RCRA/Superfund Hotline at (800) 424-9346.*

Bill Proposes Encouragement of Environmental Exports

A bill to facilitate and encourage export of U.S. environmental technologies has been introduced into Congress. The National Environmental Trade Development Act would establish a federal-private partnership, the Environmental Trade Promotion Council, to develop national strategy for increasing environmental exports; authorize funding to assess the import needs of foreign countries and provide export assistance to small U.S. companies; establish an environmental service corps within the Peace Corps; and authorize American business and environmental centers in foreign countries to assist U.S. firms, trade associations, and state economic development offices on a user-fee basis. For more information, contact *Sue Waldron of the office of the House Committee on Merchant Marine and Fisheries at (202) 225-4047.*

R0021578

Stormwater Runoff Quality Improves



Courtesy of Municipality of Metropolitan Seattle

Workers in Seattle prepare a site on Lake Union's shore for the outfall of the new dedicated stormwater sewer system.

The Municipality of Metropolitan Seattle, Wash., (Metro) is pursuing a two-part program to improve the quality of stormwater runoff from one of the city's drainage basins. Metro recently completed a source control project to maintain the low levels of contaminants in stormwater runoff and prevent inadvertent discharges and spills. It is also separating the stormwater collection system from the existing combined sewer system to eliminate combined sewer overflows (CSOs).

The two projects focus on a 688-ha (1700-ac) drainage basin in a commercial-residential area. Contaminant concentrations in the basin's stormwater runoff are typical of pollutant concentrations in urban runoff. About 500 small businesses are located in the drainage basin.

Drainage from the basin, which currently enters the sanitary sewer, will be diverted to Lake Union at the south end of the area. This diversion will send untreated stormwater runoff into the lake. To ensure that contaminants in the runoff will not exceed existing levels, Metro conducted a source control project with the following objectives:

- identify and correct illicit connections to the stormwater drainage system,
- identify business activities that might contribute pollutants to the storm drains and work with those businesses to control the release of those pollutants,
- reduce the likelihood of accidental spills of chemicals or petroleum products into the storm drains by encouraging spill control practices, and
- increase business operator awareness of the association between their activities and the introduction of contaminants into the storm drains.

A source control program and new storm sewers control runoff contamination in an urban watershed

The source control project's approach differed from a typical command and control approach. Metro educated business owners on proper waste management through site visits and direct mailing of information to encourage them to keep contaminants out of storm drains. Project personnel also published waste management articles in community and business publications and stencilled storm drains along major streets and parking lots.

Metro personnel mailed information about the sewer separation project to affected business. They also visited 80 businesses that might handle contaminants that could be discharged or spilled into the storm drainage system.

A source control project survey revealed that business operators were well-informed about basic waste management issues, employed good waste disposal practices, and had a high level of environmental awareness. Site visits, however, identified three storm drain disposal problems: soap from vehicle wash water, oily shop floor wash water, and chronic gasoline spills from fueling stations. Information on the toxicity of soaps used to wash vehicles was collected, and the effect of soapy stormwater runoff on Lake Union is being evaluated.

Information on oily water and gasoline spills was collected during site visits and alternatives to current disposal practices were developed. Metro then sent letters to all businesses with the potential to contribute to these problems. These letters identified the problem, listed acceptable alternatives, and included a list of suppliers of spill control materials. Letters sent to fuel stations offered limited assistance from Metro. The letters requested voluntary adoption of these practices.

Metro used fecal coliform and nutrient levels to detect cross-connections—sanitary lines connected to the storm drain system. The low levels of fecal coliform

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bacteria and other pollutants indicated that cross-connections were not a problem in the watershed. This conclusion was further supported when only one cross-connection was found during the 80 site visits.

Evaluating Project Effectiveness

Determining the source control project's effectiveness by direct stormwater sampling was not feasible because runoff from parking lots and roads contributes the same types of contaminants as those produced by the basin's businesses. Therefore, the project's effectiveness was measured through two other indicators.

One of the indicators was a modified cost-benefit analysis. Metro compared the cost of cleaning up a contamination source discovered during the project with the cost of the source control project itself. Metro estimated the cleanup costs of 3 years of discharge to a nearby lake to be $57,000 to $103,000. The source control project, with follow-up for 3 years, was estimated to cost $58,000. From this analysis, Metro determined that the financial benefits of the program would exceed the program's cost.

A before-after survey was used as a second indicator of the source control project's effectiveness. The survey evaluated changes in business owners' attitudes and knowledge about waste management. The survey revealed the following:

- respondents, who exhibited a high level of knowledge in the baseline survey, did not show a significant increase in knowledge;
- business owners' attitudes about the government accessibility for waste management problems improved;
- improvements in waste management practices rose from 46% to 64%;
- respondents learned about waste management regulations most readily through printed materials, on-site consultations, and information hotlines, rather than public meetings; and
- respondents perceived spills of oil, fuel, and antifreeze to be the highest risk situations, both to the environment and to business liability; businesses generally rated risks to the environment as greater than risks to business liability.

Metro Separates Pipelines

Since completing the source control project, Metro has begun work on the CSO control project. The project will separate the basin's stormwater from the existing combined sewer system by installing a new dedicated stormwater pipeline. The new pipeline is designed to reduce CSOs by 50%, to 4.2×10^6 L/yr (1.11×10^6 gal/yr). When completed, the pipeline will convey stormwater runoff by force and gravity mains to the outfall in Lake Union, which will reduce wet weather flows through the existing sanitary sewer and will reduce the frequency and volume of CSOs.

Metro's University Regulator Station governs the flow of wastewater to the area's treatment plant and sends untreated wastewater and stormwater into nearby Portage Bay when overflows occur. The regulator handles average flows of 539 L/s (12.3 mgd) and peak flows of 789 L/s (18 mgd).

The regulator station receives much of its stormwater from the Green Lake trunk sanitary sewer, which conveys stormwater runoff from three major sources: the outflow from a nearby lake, storm runoff from an interstate highway, and storm runoff and groundwater from a storm drain which collects water from the North Seattle area. The first section of the new pipeline will convey stormwater from the Green Lake trunk 1798 m (5900 ft) through a 1-m-dia (3.5-ft-dia) storm drain to a new belowground pump station. The 1972-L/s (45-mgd) pump station will be built 12 m (40 ft) below ground to reduce visual impact. The pipeline will then convey stormwater 1280 m (4200 ft) through a 1-m-dia (3.5-ft-dia) force main to a 0.9-m-dia (3-ft-dia) gravity discharge line. The gravity discharge line will extend 1554 m (5100 ft) to a 0.9-m-dia (3-ft-dia) outfall in Lake Union. The depth of the pipeline will range from 3 m (10 ft) along the force main to 9.1 m (30 ft) at the outfall.

The new storm sewer system is expected to convey 34.5×10^8 L/yr (7.87×10^8 gal/yr) of runoff from the drainage basin, plus the Green Lake trunk stormwater. By diverting stormwater runoff, Metro expects to reduce average wet weather flow to the treatment plant by 219 L/s (5 mgd).

According to a water quality impact analysis, the new pipeline will not have any adverse environmental impact on Lake Union. Dissolved oxygen levels will not be affected, nor will levels of human pathogenic viruses. While levels of some metals in the sediments around the outfall are expected to increase, this should not adversely impact the lake. The new pipeline is expected to increase phosphorus levels in the lake, but based on modelling results Metro believes these levels will not be significant.

—***Kevin Connell**, news editor*

Superfund Site Reuse Plan Unveiled

EPA recently announced that the Industri-Plex Superfund site in Woburn, Mass., will be remediated and redeveloped. The remediation plans include a new interstate highway interchange and a regional transportation center; the transportation center's parking lot will cap part of the site. Construction costs will be lower than if the center were built elsewhere because of the site's Superfund status. In addition, the site's potentially responsible parties (PRPs) are paying for site preparation, development costs, and a significant portion of the parking lot construction. In return, the PRPs will be able to open that area for redevelopment.

The new interchange will provide access to the transportation center. Future plans for the center include a 2500-car park-and-ride facility, Massachusetts Bay Transportation Authority commuter rail service, a shuttle services to Logan Airport and possibly a heliport. The park-and-ride facility will satisfy portions of the state's environmental commitments to the $7 billion Central Artery/Third Harbor Tunnel highway project. The facility will provide out-of-city parking and mass transportation to reduce the number of vehicles coming into Boston and ensure compliance with the Clean Air Act.

Site Has Long History

The Industri-Plex site includes streams and ponds, waste deposits, and active and inactive commercial facilities. Approximately 24 ha (60 ac) of the 99-ha (245-ac) site are currently used for commercial enterprises; the balance of the property is undeveloped. From 1853 to 1969 the site was occupied by manufacturing facilities that produced lead and arsenic insecticides; produced chemicals for local leather, textile, and paper industries; and manufactured glue by cooking raw and waste-chromium-tanned animal hides. Chemical wastes and hide residues were buried throughout the property.

In the 1970s, construction activity uncovered the industrial by-products and wastes that had accumulated on the site during the previous 120 years. The exposure and movement of decaying hides released noxious odors, and citizens com-

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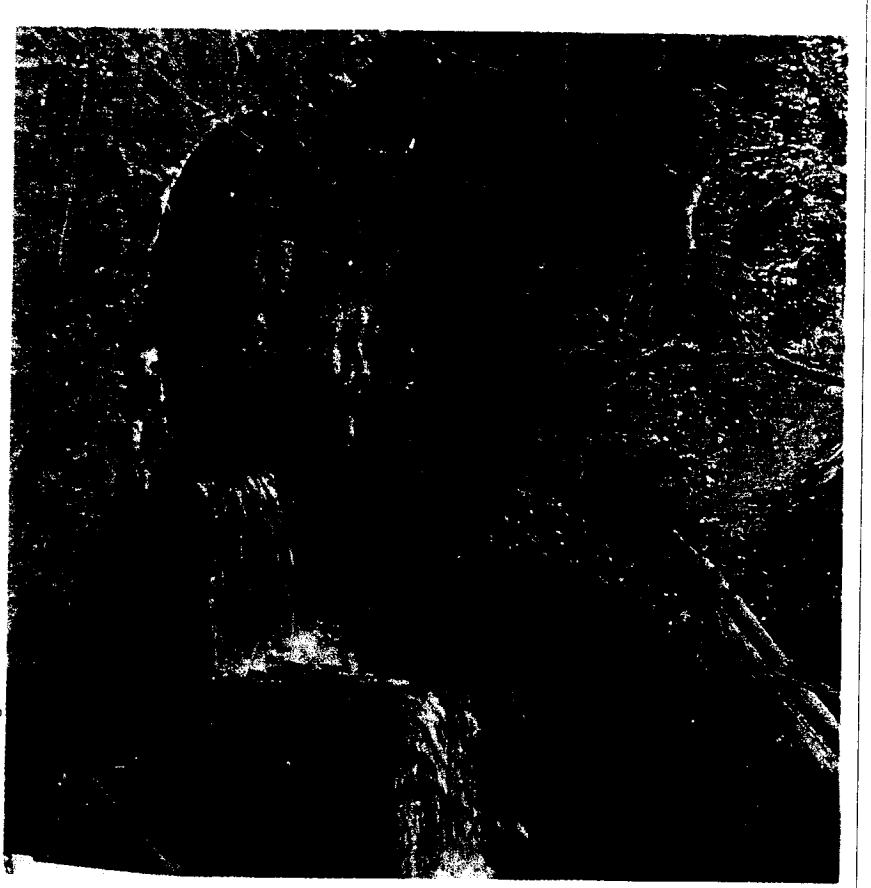
EPA Expands Stormwater Control Permitting



Courtesy of Rob Goldberg

Urban runoff from storm sewers, like this one in Philadelphia's Fairmount Park can contribute, contaminants to urban streams.

By the end of 1993, EPA will require light industry without exposure and small construction sites to obtain permits for their activities to ensure that stormwater runoff is controlled. These new rules come in response to a lawsuit by the Natural Resources Defense Council (NRDC), which challenged EPA's exclusion of the two industry categories from stormwater regulations.

The NRDC law suit included several complaints regarding the agency's planned management of the stormwater permitting process. The U.S. Court of Appeals held that most of EPA's stormwater program was valid, but agreed with NRDC's objections over the two exemptions. The court invalidated EPA's exclusion of light industries whose activities are not exposed to rain, and construction sites smaller than 2 ha (5 ac).

Two industry categories previously excluded from EPA's stormwater rules are now regulated

The court eliminated the light industry exemption, finding it "arbitrary and capricious" because of the agency's different rules for stormwater control for heavy and light industries. EPA required proof of proper stormwater management from heavy industry through permitting, but did not require permits for light industries without exposure.

Similarly, the court invalidated the exemption for small construction sites because EPA could not prove that "construction activities on less than 5 acres are non-industrial in nature."

Current Regulations

EPA's stormwater regulations were promulgated to improve stormwater runoff quality. EPA estimates that at least 33% of all contamination in lakes and estuaries and 10% of all river contamination are caused by stormwater runoff. Surface water receiving large volumes of urban runoff frequently fails health standards for human contact because of the high levels of fecal bacteria from animal feces and illicit connections.

Ongoing testing in Philadelphia's Wissahickon Creek, for example, has revealed summertime fecal coliform levels 300% greater than the human safety standard, even though a significant portion of the watershed is a forested park. Additionally, the Philadelphia Water Department tested its 484 storm sewers to apply for Part I of its stormwater permit and found that water flowed out of 199 of the outlets during dry weather. Many of the dry weather flows contained extremely high fecal coliform levels—an indication of illicit discharges of untreated wastewater.

According to a study by the NRDC, the total pollutant load from urban runoff may exceed that from industrial discharges in many cities. EPA indicates that construction activity without stormwater control measures can release 25 to 31 $g/m^2 \cdot d$ (220 to 275 $lb/ac \cdot d$); a 40-ha (100-ac) development could potentially contribute more than 4500 Mg (5000 tons) of sediment to receiving water bodies each year.

EPA has required large construction sites and other industries to obtain stormwater control permits for their activities. Currently, the following industry categories must apply for some type of stormwater permit:

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- manufacturing industries;
- light industry with exposure;
- recycling facilities;
- transportation facilities;
- wastewater treatment plants;
- landfills;
- construction sites larger than 2 ha (5 ac);
- steam electric-power-generating facilities;
- hazardous waste treatment, storage, and disposal facilities; and
- mining, oil, and gas industries.

The stormwater regulations require these industries to characterize the runoff from their sites and institute practices to prevent this runoff from collecting contaminants. These activities are monitored through permitting.

EPA's stormwater regulations allow many permittees to submit an interim Notice of Intent (NOI) that demonstrates a willingness to use best management practices such as improved methods for handling and storing materials and preventing spills. Some permittees, including waste disposal sites, facilities with coal piles, and the metals industry, must also perform semiannual or annual stormwater monitoring.

Permitting Varies

Stormwater-control permit requirements vary between states. In nondelegated National Pollutant Discharge Elimination System (NPDES) states, many dischargers can file an NOI under EPA's general permit. In delegated NPDES states with an existing general permit, dischargers may be able to submit an NOI under that state's general permit. In a delegated state with no general permits in place, however, a regulated facility must file a more costly individual permit. Some of these states without the authority to issue general permits are in the process of obtaining this authority.

The permit application deadline for ongoing activities was October 1, 1992. Facilities engaging in new activities that will contribute contaminants to stormwater runoff generally must submit an NOI at least 2 days before commencing the activity. Pollution prevention plans must be implemented by October 1, 1993.

—***Rob Goldberg***, *Environmental Associates of the Academy of Natural Sciences, Philadelphia, Pa.*

Moynihan Bill Sets Goals

Believing that "science can lend much needed coherence, order, and integrity to costly and controversial" decisions about environmental priorities, Sen. Daniel Patrick Moynihan (D-N.Y.) has introduced Senate Bill 110: the Environmental Risk Reduction Act of 1993. Based on the premise that "environmental protection activities...shall attain the greatest risk reduction possible with the resources available," the bill proposes to require the following:

- accurate, quantitative estimates of human and ecosystem exposure to all important risk factors;
- accurate techniques for predicting the effects of that exposure;
- an adequate understanding of technical, economic, social, and legal alternatives for reducing exposure to risk factors; and
- accurate estimates of the costs and benefits of alternatives for reducing risks.

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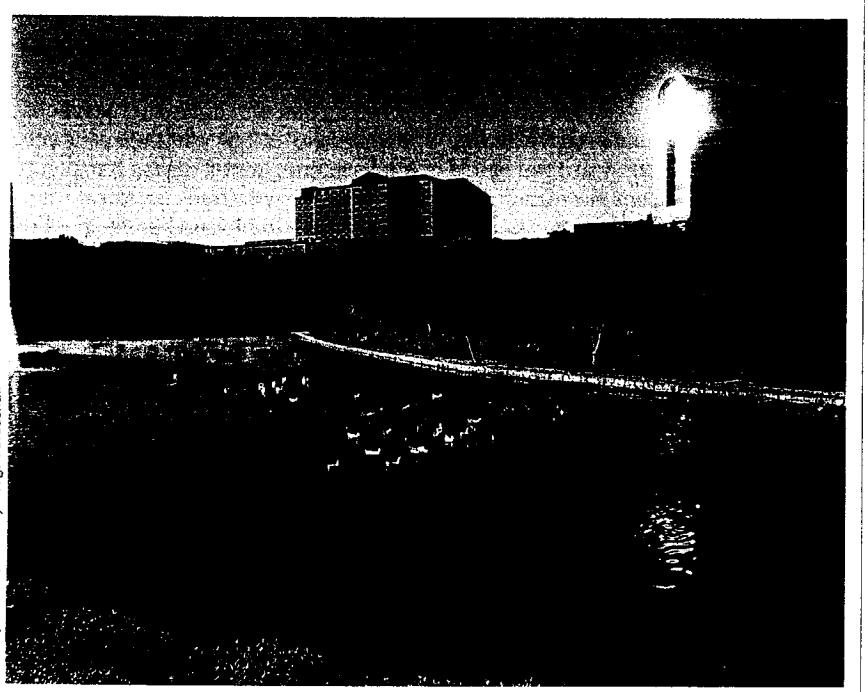
Promoting Consistent Stormwater Management



Courtesy of CDM; Photo by Reagan Bradshaw

Stormwater control measures, such as this detention pond protect local water quality.

The California Stormwater Quality Task Force recently released a best management practices (BMP) handbook to help the regulated community select control measures to reduce stormwater pollutant discharges. The handbook provides general guidance for municipalities, industries, and construction sites to develop and implement BMPs in stormwater management programs.

The task force was formed by the State Water Resources Control Board and the American Public Works Association to help the state implement the stormwater program mandated by the National Pollutant Discharge Elimination System (NPDES). The task force helped the state board draft the general NPDES permits and continues to address regulatory and legislative issues, both in California and nationwide.

A handbook is available for NPDES permittees to make informed decisions on stormwater control BMPs

Statewide Cooperation

The handbook was funded by the state, cities, and special flood control districts in California. Many cities and special districts were deciding what to do with their own stormwater programs and were willing to contribute funds to a project that would foster consistency in BMPs developed in different districts.

The Task Force hired a consultant team and appointed an 11-member technical advisory committee (TAC) to guide the handbook's development. The TAC included three representatives from municipalities, three from the construction industry, three operators of industrial activities, and two representatives of regulatory agencies. The TAC members provided a user-oriented focus, contributing personal experiences and identifying concerns about BMPs that might unduly disrupt their activities.

The handbook consists of three volumes, each directed toward a specific audience. The first volume, the Municipal BMP Handbook, addresses municipal stormwater quality management as required under the NPDES municipal permit program. The second volume, the Commercial-Industrial BMP Handbook, addresses stormwater quality management for facilities that currently are covered, that eventually will be covered, or that are similar to facilities covered by an NPDES general permit for industrial activities. The third volume, the Construction BMP Handbook, addresses erosion control and other stormwater quality management plans required under an NPDES general permit for construction activities.

The overall goal of a stormwater management program is to reduce the discharge of pollutants. The handbook is organized to help the user develop and implement such a program. Each volume of the handbook includes chapters on the following topics:

- pertinent regulations regarding the NPDES permit, who must get a permit, and fundamental technical information about stormwater pollutants and their sources;
- how to develop a stormwater management program or stormwater pollution prevention plan;
- guidance on the selection of BMPs for the plan;
- details of source control BMPs;
- details of treatment control BMPs; and
- guidelines for measuring BMP performance.

The handbook includes information adapted from other BMP manuals to suit the unique conditions and needs of

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California. The state has widely varying rainfall volumes (from 25.4 to 2540 mm/yr [1 to 100 in/yr]), topographic features (including the highest and lowest points in the U.S.), and soils. Water quality regulations contignons are administered by nine regional water quality control boards because of the state's size, diversity, and population.

The handbook is not intended to dictate selection of BMPs, as this is done by the permittee, but to provide the framework for an informed selection of BMPs. The handbook primarily addresses the requirements of the stormwater program set forth in section 402(p) of the Clean Water Act. It should also be useful for ensuring compliance with the emerging coastal nonpoint pollution control program mandated under the Federal Coastal Zone Act Reauthorization Amendments of 1990.

The handbook discusses other federal and state programs that require implementing BMPs similar to those that control pollutants in stormwater discharges. The authors hope that this discussion will aid the regulated community in meeting requirements of several programs with an integrated set of BMPs. Examples include the development of integrated BMPs for household hazardous waste collection, used oil recycling, spill prevention and response, land development policies and reviews, and air quality management.

Format Is User-Friendly

The authors were determined to make the manual user-friendly because the information would not be valuable if it could not be easily understood. Project director Dr. Larry Roesner notes, "What makes this handbook so unique is that while there are bits and pieces in the existing literature of how to develop the best management practices, this handbook pulls it all together. It is one reference that addresses not only who has to get a permit and how to develop a storm water management program, but also which BMPs might be used, including guidance for prioritizing these BMPs."

The handbook incorporates the following user-friendly concepts:

Three Volumes. The content of each volume is directed toward a particular audience (municipal, industrial-commercial, or construction), addressing only issues of interest to that audience.

Standard Format. Each of the three volumes is organized the same way to make it easy for one audience to refer to a volume intended for another audience.

Guidance on BMP Planning. By focusing on a long-range planning process, the handbook advises users to prioritize control measures and use their own creativity in determining a feasible, cost-effective, phased control plan.

Fact Sheets. Discussions of the BMPs are presented on fact sheets (1-to-2-page overviews with additional information where needed) and organized by activity categories to ease the selection process. Each fact sheet describes either suggested BMPs for a particular activity that causes pollution (such as vehicle maintenance) or a specific activity or facility which is a BMP (such as street sweeping). Users can copy the fact sheets, modify them as appropriate, and insert them into pollution prevention plans, employee training programs, and educational materials.

Three-Ring Notebook Format. The handbooks are designed as "living documents," with removable pages for copying and distribution, and space for changes and expansion.

The effect of this handbook on stormwater management in California depends on how aggressive the state becomes in implementing the program. The authors foresee a real benefit in the consistent selection of BMPs, particularly to the construction and industry sectors.

—***John Aldrich***, *Camp Dresser & McKee Inc., Walnut Creek, Calif., and* ***Malcolm Walker***, *Larry Walker & Associates, Davis, Calif.*

Can Demand-Side Management Save Money?

Demand-side management (DSM), long recognized by large electricity consumers as a way to reduce their energy costs, is becoming common among a broad range of medium-sized consumers, including wastewater treatment plants. It offers treatment plants an opportunity to reduce operating costs and acquire energy-efficient equipment at a significantly reduced capital cost.

Any technique used to reduce the demand for or use of electricity is called DSM. These techniques include installing more energy-efficient equipment and shifting existing energy demand from a utility's peak periods to its off-peak periods. Electricity consumers benefit from reduced electricity bills, and the electrical utility benefits from reduced need to operate peak-use generators (usually its least efficient) or build new power plants.

Utilities Offer Subsidies

While energy conservation has always reduced consumers' electricity bills, utilities' DSM programs frequently offer consumers greater financial benefits than previously available. Many utilities are now willing (or are being compelled by state regulatory agencies) to pay customers subsidies for reducing demand and saving energy. Under the new National Energy Policy Act, tax benefits may also be available for the installation of DSM measures.

Through DSM, wastewater treatment plants can reduce their monthly utility bills and may be able to use the tax benefits and utilities' subsidies to help pay for energy-efficient equipment that they have installed. In addition to reducing electricity consumption, new equipment may increase a plant's productivity.

How Do DSM Programs Work?

A new service industry has emerged to help medium-sized electricity consumers take advantage of DSM's benefits. While large consumers may deal directly with electrical utilities, medium-sized consumers can seek the expertise of an energy services company (ESCO), which acts as an intermediary between the utility and the consumer. The ESCO will conduct an energy audit of a consumer's plant or building, recommend energy reduction strategies, provide the necessary monitoring data to the electrical utility, and may install and maintain DSM equipment.

An ESCO also may finance much of the capital cost of the new equipment at a facility in return for the utility subsidy and sometimes a percentage of the consumer's utility bill savings. The consumer acquires new equipment with little or no capital outlay or risk, realizes a portion of the energy savings, and reduces equipment operating and maintenance costs.

Implementing DSM Contracts

When a wastewater treatment plant uses an ESCO to implement DSM measures, two contracts typically are involved: a contract between the utility and the ES-

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Regulating Metal Toxicity in Stormwater

A model to predict the form and toxicity of metals could support stormwater quality regulations

Cynthia Paulson
Gary Amy

The best way to regulate metals in stormwater runoff has been widely discussed. Historically, instream quality criteria for metals, including EPA criteria (1986 and 1987 revisions), have addressed total metal concentrations. New EPA guidance on metals allows for the regulation of bioavailable metals, which make up a fraction of the total concentration. Predictive modeling of the form and associated toxicity of metals under various conditions would improve data transferability and supplement the proposed EPA approaches to metals regulation. EPA collaborated with the U.S. Geological Survey and local agencies to study stormwater runoff quality in 28 cities as part of the Nationwide Urban Runoff Program (NURP). Results showed that total metal concentrations frequently exceeded acute instream criteria set for aquatic life. These conclusions may be misleading because the metals that make up total concentrations are not always in bioavailable forms.

In the absence of sufficient data on metal form and toxicity, total concentrations provide a conservative measure. EPA criteria were based on laboratory toxicity tests performed in relatively particulate-free waters. This suggests that the criteria are more appropriately applied to dissolved, rather than total, metal concentrations.

Total metal concentrations include several forms: particulate and dissolved, inorganically or organically complexed (called *bound*), and free (see Figure, *Metal Forms and Toxicity*). Bioavailability and toxicity vary with the form of the metal. The most bioavailable forms that are toxic to aquatic life are free metals and some weak inorganic complexes. Metals in the environment, particularly in stormwater runoff, are usually present in particulate and strongly complexed (organic) forms, with only a small fraction of the total metal concentration being bioavailable.

Over time, with changing chemical conditions, metals that are bound in nonbioavailable forms can dissociate and become bioavailable. For example, particulate metals can be released from sediments. Therefore, in water quality planning, it is crucial to consider the possibility of metals becoming bioavailable over the long term, in addition to the short-term considerations discussed here.

EPA INTERIM METALS GUIDANCE

To clarify the metals issue, EPA released the draft guidance in May 1992 on interpreting and implementing aquatic life criteria for metals; the agency plans to issue the final guidance sometime in 1993. The

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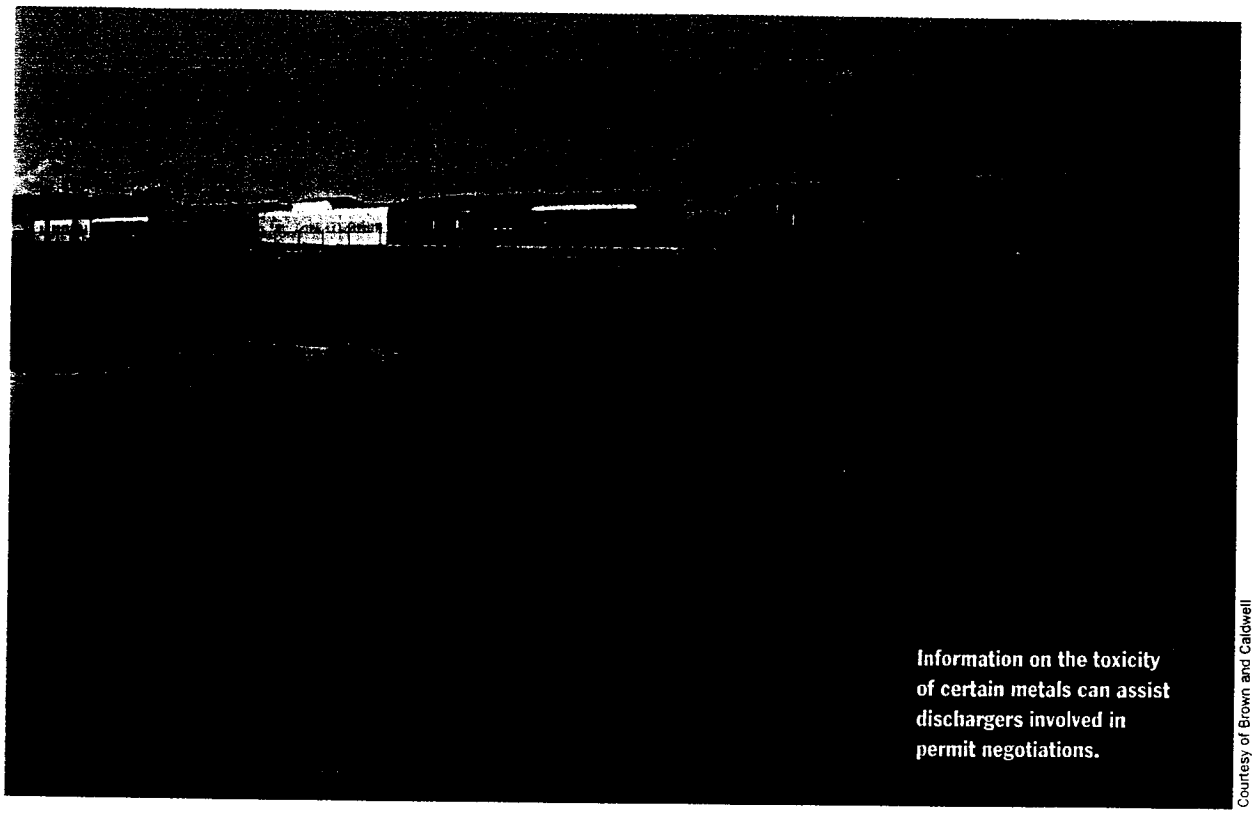


Information on the toxicity of certain metals can assist dischargers involved in permit negotiations.

Courtesy of Brown and Caldwell

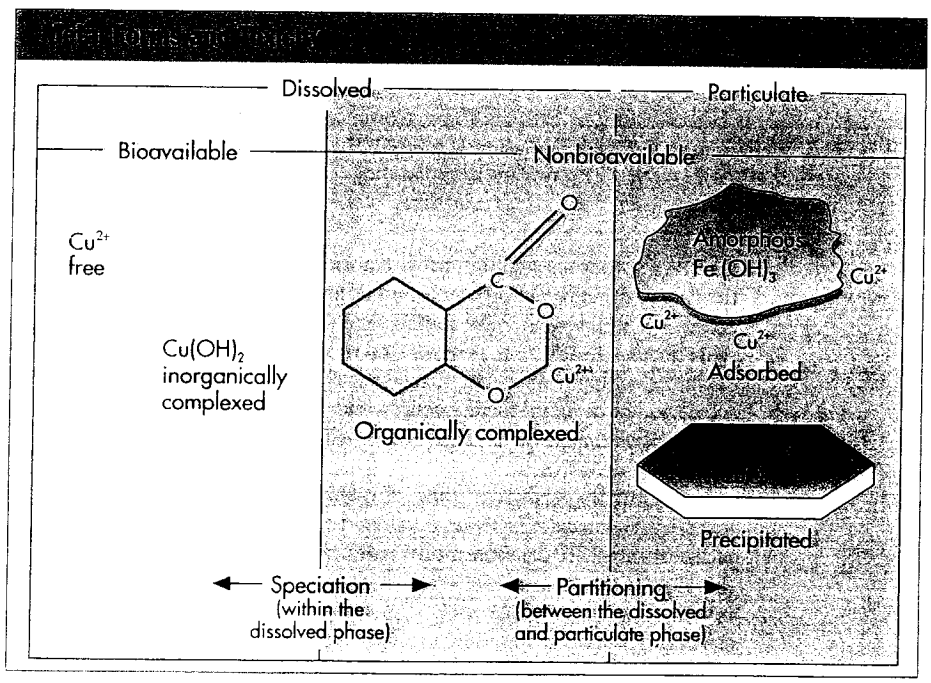
interim guidance recognizes that regulating total metal concentrations may be overly protective, and allows for consideration of the bioavailability of metals to aquatic life. The guidance provides three alternative approaches:

- continuing current practices of comparing total concentrations to existing criteria,
- comparing dissolved metal concentrations to criteria and translating dissolved values into total allowable effluent concentrations, and
- using biological testing to compare toxicity of metals in receiving waters to laboratory test waters (the water effect ratio).

A major drawback of the proposed EPA alternatives is that the approaches rely on site-specific data to describe metal toxicity effects, without a basis for explaining the mechanisms (metal partitioning and speciation) that influence the toxicity. Furthermore, none of the approaches can be easily transferred from one chemical environment (site) or chemical condition (time) to another.

A PREDICTIVE MODEL FOR METALS REGULATION

A geochemical model (for example, the EPA model MINTEQA2) capable of predicting the form and associated toxicity of metals under various conditions could be an effective tool for supporting metals regulation. Combining a modeling approach with one of



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Chemical Processes Controlling Partitioning and Speciation

The distribution of total metal concentrations among different metal forms, with associated toxicity, can be defined as *partitioning* (the distribution of metals among particulate and dissolved phases) and speciation [the distribution of dissolved metals among inorganic or organic complexes (bound forms) or free forms]. The partitioning and speciation of metals vary by metal and site-specific conditions. Several chemical processes control partitioning and speciation in stormwater: adsorption, inorganic complexation, organic complexation, and solid precipitation-dissolution (that is, over- or undersaturation).

Adsorption. **The adsorption of metals onto particulates is important in waters with high concentrations of suspended solids, such as stormwater runoff. Many different kinds of solids can serve as an adsorbent surface, including iron and manganese hydroxides, clays, aluminum oxides, silica, and solid organics. Amorphous iron oxide is frequently used as a model solid to evaluate adsorption of metals because it is often the predominant adsorption surface and a well-developed database exists. The order of preference for adsorption sites on a number of surfaces has generally been shown to be lead, copper, and zinc, in decreasing order.**

Inorganic complexation. **Inorganic complexation occurs when complexes are formed between metal cations and a number of common anions, including hydroxide (OH^-), carbonate (CO_3^{2-}), sulfate (SO_4^{2-}), and chloride (Cl^-). The formation of inorganic complexes depends on pH conditions as well as on concentrations of formation species and competing species. Inorganic complexes are generally not very strong complexes; and they are less likely to form, unless anion concentrations are high.**

Organic complexation. **In organic complexation, metals form strong bonds with organic ligands such as humic and fulvic acids, which are a function of dissolved organic carbon (DOC). In general, the relative strength of organic complexes with metals follows the decreasing order: lead, copper, zinc, manganese (Mn), calcium (Ca), magnesium (Mg). However, high concentrations of Ca^{2+} and Mg^{2+} can compete with trace metals for organic binding sites. Hydrogen ions also compete with metals for organic complexation sites, so that organic complexation capacity for metals decreases with decreasing pH.**

Solid precipitation-dissolution. **In solid precipitation-dissolution, oversaturated solids (when concentrations of solids exceed solubility products) could possibly precipitate from stormwater, given sufficient time. However, storm events are short-term in nature (lasting less than 8 hours) when compared to the time required for solids to precipitate (days or weeks). As a result, oversaturated solids in stormwater will generally remain oversaturated or will precipitate to only a very minor degree because of kinetic limitations. At this time, data on solid precipitation kinetics do not support incorporating solids formation into stormwater modeling.**

the EPA approaches would improve transferability of the data to other receiving waters and other chemical conditions and would make metals regulation more cost effective. In addition, geochemical modeling can explain the chemical mechanisms that control dissolved and bioavailable metal fractions and can predict the effects of changing water quality, as well as various regulatory scenarios.

A preliminary predictive model of metal form and toxicity in stormwater has been developed on the basis of historical NURP data (the model is described in further detail in a related paper accepted for publication in *Water Environment Research*). However, to understand the model, it is important to review the key chemical processes that control metal form and associated bioavailability (see Box, *Chemical Processes Controlling Partitioning and Speciation*).

The NURP data from Fresno, Calif., and Salt Lake City, Utah, along with summary NURP data, were reevaluated through statistical analysis and geochemical modeling to:

- characterize stormwater-quality matrices, with particular focus on factors that could affect metal form;
- define key processes and factors that control the partitioning and speciation of metals in stormwater; and
- compare metal concentrations in stormwater runoff to acute instream criteria to test the initial conclusion from the NURP studies that stormwater metals could threaten aquatic life.

The two cities from the NURP study are similar in size, with populations between 150,000 and 250,000. Rainfall characteristics are also similar, including rainfall amounts [280 to 610 mm/yr (11 to 24 in./yr)], average intensity [0.63 to 0.76 mm/hr (0.025 to 0.030 in./hr)], and number of storms (60 to 65/yr). Residential, commercial, and industrial land uses were evaluated at both sites. Key differences between the two locations include soils (Fresno's soil consists of granitic alluvium; Salt Lake City's, mixed sediments) and average drainage basin size [Fresno basins averaged about 0.5 km^2 (0.2 mi^2), one-tenth the size of Salt Lake City basins, which were about 5 km^2 (2 mi^2)].

Stormwater and river water quality have several key differences. Although stormwater quality is highly variable, mean stormwater conditions were evaluated to determine key differences between typical stormwater and river water that may affect metal form and toxicity (see Table, *Comparison of Stormwater and River Water Quality*).

Constituents with higher concentrations in stormwater than in average river water (nonstorm event) included suspended solids (3 to 6 times); dissolved organic carbon (DOC) (3 to 5 times); ammonia (5 to 20 times); dissolved phosphorus (10 to 100 times); and metals, including manganese, iron, copper, lead, and zinc (5

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Comparison of Stormwater and River Water Quality

Constituent	Fresno[a]	Salt Lake City[a]	NURP[b] median/90th percentile	Average river[c]
Temperature, °C	–	15		4–25
Conductivity, µS	104	440		110
pH	7.0	7.7		7.5
Total suspended solids, mg/L	180	80	100/300	30
Dissolved organic carbon, mg/L	50[d]	14		5
Hardness, mg/L $CaCO_3$	30	240		50
Alkalinity, mg/L $CaCO_3$	30	140		52
SiO_2, dissolved mg/L	3	11		10
Ammonia and Ammonium ion, dissolved mg/L N	2	0.5		0.1
NO_3^-/NO_2^-, dissolved mg/L N	0.8	0.9	0.7/1.8	1
Phosphorus, dissolved mg/L	0.9	0.2	0.3/0.7	0.02
Calcium, dissolved mg/L	7	40		13
Magnesium, dissolved mg/L	2	20		3
Sodium, dissolved mg/L	15	50		5
Potassium, dissolved mg/L	6	6		1
Chlorine, dissolved mg/L	20	50		6
SO_4^{2-}, dissolved mg/L	10	70		8
Manganese, total µg/L	260	90		6
Iron, total and (dissolved) µg/L	3900(173)	2400(29)		1100(55)
Copper, total and (dissolved) µg/L	34(8)	34(9)	34/93	5(1)
Lead, total and (dissolved) µg/L	120(14)	90(7)	144/350	2(0.2)
Zinc, total and (dissolved) µg/L	270(100)	130(24)	160/500	25(10)

[a]Values for Fresno and Salt Lake City sites represent geometric means of the data for all Nationwide Urban Runoff Program (NURP) sites evaluated in those cities.

[b]NURP data represent event mean concentrations (total metal load divided by total stormwater runoff volume) for the median and 90th percentile (equal to or greater than 90% of the data) urban sites from all sites across the U.S. (EPA, 1983). Phosphorus data from the NURP summary represent total concentrations.

[c]References for average river water quality (nonstorm event) include Meybeck (1979), *Concentrations des Eaux Fluviales en Elements Majeurs et Apports en Solution aux Oceans*, Revue de Geologie Dynamique et de Geographic Physique (Paris), 21:215-246 (major constituents); Livingstone (1963), *Chemical Composition of Rivers and Lakes: Data of Geochemistry* (6th ed.), USGS Survey Professional Paper 440-G, pp. G1-G64 (nitrate); Thurman (1985), *Organic Geochemistry of Natural Waters*, Nijhoff and Junk Publishers [Dissolved organic carbon (DOC) and POC (particulate organic carbon)]; and Salomons and Forstner (1984) *Metals in the Hydrocycle*, Springer-Verlag Publishers (dissolved metals). Values for temperature, total metals, pH, suspended solids, ammonia, and phosphorous are estimated based on typical observed values.

[d]Fresno DOC concentration represents the geometric mean of reported data. Because 50 mg/L is believed to be higher than actual, a more realistic 15 mg/L, observed in related studies by the authors, was used for DOC in modeling.

to 50 times). The pH conditions for stormwater and river water were generally similar.

Dissolved and bioavailable metals comprise a small fraction of total concentrations. Although the variability in storm event characteristics causes a high degree of variability in stormwater quality, the relationship between dissolved and total concentrations of metals was relatively consistent. Coefficients of variation (the standard deviation divided by the mean) were 50% to 70% for the dissolved metal fractions, compared with 50% to 150% or more for most other stormwater constituents.

Measured dissolved fractions at the Fresno and Salt Lake City sites constituted 25% or less of the total metal concentrations for copper and lead, and were 20% to 40% for zinc (see Figures, *Comparison of Total, Dissolved, and Bioavailable Metals to Acute Criteria* for Fresno and Salt Lake City). Bioavailable metal concentrations (defined here as free and inorganically complexed metals) were predicted using an equilibrium model (MINTEQA2) that was adapted to the Fresno and Salt Lake City data. Based on the model predictions, the bioavailable fraction constituted 6% or less of total copper and lead concentrations and about 10% to 35% of total zinc concentrations at the Fresno and Salt Lake City sites.

Metals can be discharged to stormwater in a mineral-particulate or dissolved form. As a conservative approach for the purpose of modeling, however, no particulate metals were assumed initially, and the entire metal concentrations were assumed to be initially present in a dissolved form. Thus, total metal concentrations were used as input values for copper, lead, and zinc, and the model was allowed to predict partitioning and speciation. (Although chemical conditions in stormwater runoff are highly variable and are not likely to reach equilibrium before entering a receiving water, equilibrium was assumed for modeling purposes.)

Four key factors control the form of metals. Statistical analysis and geochemical modeling of the

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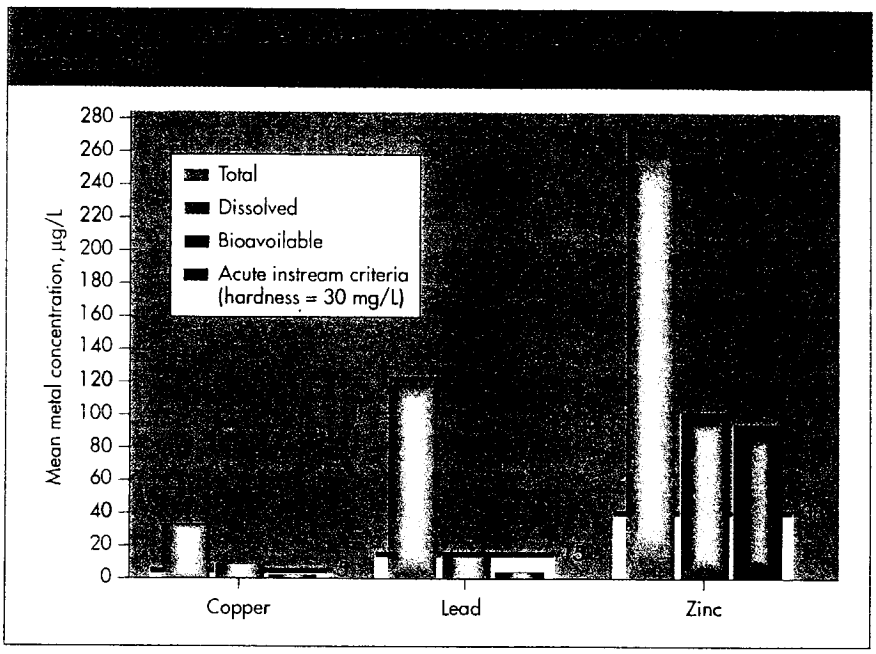

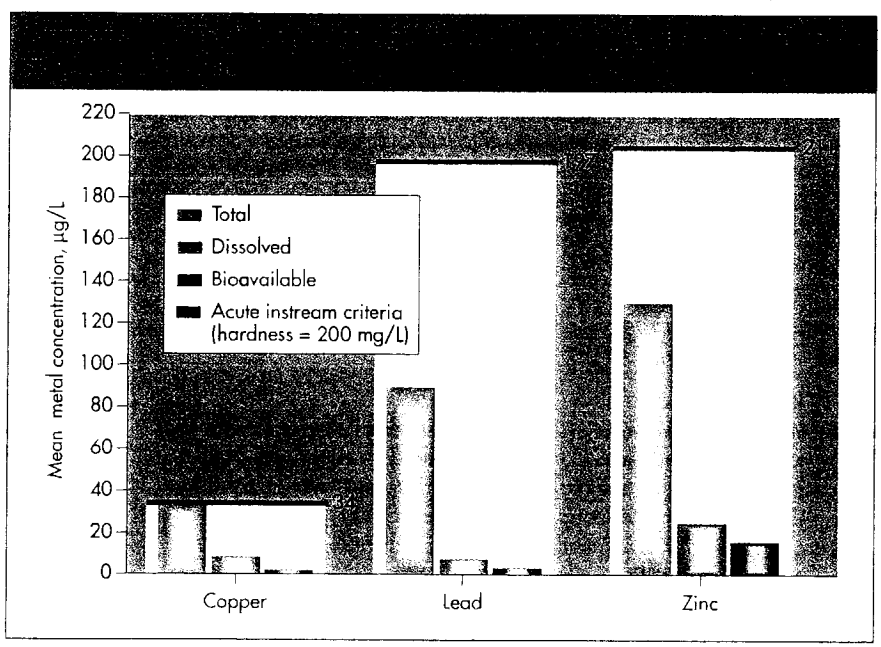

NURP data indicated that the four most important factors controlling the distribution of metals among dissolved and bioavailable fractions include *suspended solid types and concentrations, pH, total metal concentrations,* and *dissolved organic carbon concentration and character.*

Typically, as suspended solid concentrations increase, adsorption of metals onto solids also increases due to a greater number of sorption sites. In addition, higher concentrations of mineral-particulate metals may be correlated with suspended solids, perhaps related to high-flow events. The result of these two effects is a smaller dissolved fraction of copper, lead, and zinc.

Changes in pH affect organic complexation to a limited degree and strongly affect adsorption processes. In the absence of competing processes, the fraction of total metals that are adsorbed is virtually nonexistent at pH levels of 5 to 6, while virtually complete adsorption occurs at pH levels of 7 or 8. As adsorption increases with pH, the remaining dissolved and bioavailable fractions of metals decrease.

At high total metal concentrations, competition among the metals (copper, lead, and zinc) for complexation or adsorption sites can result in greater dissolved and bioavailable metal fractions for the less competitive metals, particularly zinc.

The DOC can also play an important role in determining both the dissolved and bioavailable metal fractions. Organic complexation sites on DOC compete with adsorption sites on solids to maintain metals in a dissolved rather than particulate form, thus increasing the dissolved fraction. At the same time, however, the metals that are organically complexed to DOC are not considered bioavailable. Thus, organic complexation, while increasing the dissolved fraction, decreases the bioavailable fraction. Competition from calcium and magnesium cations (Ca^{2+} and Mg^{2+}) for organic complexation sites can become significant at high metal concentrations and low DOC concentrations. Model predictions show that Ca^{2+} and Mg^{2+} could occupy 10% to 50% of the high energy organic complexation sites in the Fresno and Salt Lake City samples, respectively.

Concentrations of inorganic species (for example, OH^-, CO_3^{2-}, SO_4^{2-}) have a less significant effect in determining dissolved and bioavailable fractions of total metal concentrations. Inorganic species are less significant primarily because metal concentrations, rather than inorganic species concentrations, are limiting factors controlling inorganic metal complexation.

Dissolved and bioavailable metal concentrations exceed acute criteria to a lesser degree than total metal concentrations. To evaluate the potential impact of stormwater metals on receiving-water

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ecosystems, metal concentrations were compared to instream aquatic life criteria. For the comparison, acute criteria were calculated on the basis of reported hardness values (high hardness offsets metal toxicity) using equations from EPA's *Quality Criteria for Water* (1986 and 1987 update).

Applying instream criteria to stormwater concentrations is generally a conservative measure, because metal concentrations in stormwater runoff are typically diluted in receiving streams where criteria would actually apply. Differences in receiving water chemistry during a storm event versus stormwater runoff chemistry could also affect metal partitioning and speciation. Ultimately, a predictive model should include water quality differences between stormwater runoff and receiving waters to evaluate the impact of stormwater metals. The preliminary model used in this study focused on metal partitioning and speciation in stormwater runoff before mixing with a receiving water.

The NURP summary data on metal concentrations, along with a range of acute criteria based on the range of hardness values in the NURP data, provide a national framework (see Table, *Comparison of Total Metal Concentrations and Acute Criteria*). The summary NURP data and the data for Fresno and Salt Lake City indicate that metal concentrations in stormwater typically follow the descending order: zinc, lead, copper. Total zinc concentrations in stormwater are typically on the order of 10^{-6} *M* and total copper and lead concentrations are on the order of 10^{-7} *M*. The relative toxicity of these metals to aquatic life, based on short-term *in-situ* metal uptake experiments performed in another study, follows an order that differs from the order of occurrence: copper, zinc, lead.

Based on these data and our analysis, copper, the most toxic of the three metals, is typically present in the lowest concentrations. Zinc, which is somewhat toxic, is present in the highest concentrations. In addition, dissolved and bioavailable fractions of total metal concentrations are generally larger for zinc than for copper or lead. As a result, metal toxicity from zinc may be more significant than either copper or lead toxicity.

For comparison to criteria at the Fresno and Salt Lake City sites, geometric means were calculated from measured concentrations of total and dissolved metals in grab samples, and bioavailable concentrations were predicted with the MINTEQA2 model. Dissolved and bioavailable metal concentrations exceeded criteria much less frequently, and to a lesser degree, than total concentrations at both sites. Predicted bioavailable metal concentrations were well below the acute criteria in most cases.

Comparison of Total Metal Concentrations and Acute Criteria

Data source	Total recoverable metal concentration, µg/L[a] Copper	Lead	Zinc
NURP data			
EMC[b], median urban site	34	144	160
90th percentile[c], median urban site	93	350	500
Acute criteria[d]			
Hardness = 50 mg/L $CaCO_3$	9	34	65
Hardness = 100 mg/L $CaCO_3$	18	32	117
Hardness = 200 mg/L $CaCO_3$	34	197	211

[a]Data from results of the Nationwide Urban Runoff Program (NURP) (EPA, 1983).

[b]EMC represents event mean concentration over a storm event.

[c]90th percentile represents the value equal to or greater than 90% of the data.

[d]Acute metals criteria were calculated as a function of hardness from the equations in the EPA *Quality Criteria for Water* (1986, and 1987 update).

Implications for NPDES Dischargers

The results obtained with the model for predicting partitioning and speciation of copper, lead, and zinc in stormwater runoff hold several implications for dischargers under the National Pollutant Discharge Elimination System (NPDES) program.

- **The short-term impact of metals in stormwater runoff, based on the bioavailable fraction of total metals, is significantly less than initially concluded from the NURP studies, which were based on total concentrations.**
- **Incorporating bioavailability considerations into the stormwater metals permitting process can provide dischargers with significant savings in potential metal treatment and removal costs.**
- **A predictive model can be used to improve transferability of data and to supplement the proposed EPA approaches to metals regulation.**
- **The predictive model, which has been applied to stormwater runoff, can be extended to receiving water environments for comparing instream criteria and evaluating potential impacts on aquatic life.**

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FUTURE APPLICATION OF PREDICTIVE MODELS TO STORMWATER METALS

The MINTEQA2 equilibrium model has been applied to historical NURP stormwater data to predict partitioning with future refinements, and speciation of copper, lead, and zinc in stormwater runoff. The model could be applied as a cost-effective tool to predict bioavailable concentrations over a wide range of water quality conditions in support of the National Pollutant Discharge Elimination System (NPDES) permitting process. ■

Cynthia Paulson *is a senior engineer with Brown and Caldwell in Denver, Colo., currently on academic leave; and* ***Gary Amy*** *is a professor of environmental engineering at the University of Colorado in Boulder, Colo.*

Wetlands For Stormwater Treatment

Developers of a residential site on Lacamas Lake (Wash.), used a wetland system to treat stormwater runoff from the lake watershed.

A four-year water quality monitoring program has shown that this system is an effective biofilter that reduces nutrient and sediment loading to the lake. Many issues must be addressed during the design and implementation of a wetland system, including permitting, water quality standards, treatment performance, cost, maintenance, and expected treatment life.

An innovative wetland design filters stormwater and reduces phosphorus loading to adjacent lake

Mark F. Bautista
N. Stan Geiger

The Lacamas Lake watershed, which includes the adjoining Round Lake, is located just north of Camas, Wash., 19 km (12 mi) west of Vancouver, Wash. The availability of sanitary sewers for the Lacamas Shores development eliminated the potential for contamination of the lake by septic tank systems. However, there was still an increased volume of runoff and concomitant pollutant load from the development during storm events. Municipal storm sewers were not available to receive runoff from the development, so runoff would have to be discharged to the lake.

As part of the permitting process, a drainage plan was drafted to indicate how stormwater runoff would be collected and discharged from the site. The Washington Department of Ecology required that the quality and quantity of stormwater runoff from the development could not exceed predevelopment conditions. Therefore, runoff discharged to the lake had to be treated and detained in an on-site facility before discharge.

Runoff from the watershed provides the majority of water flowing into the lakes. Except for the shoreland area adjacent to the lakes (which accounts for 9.5% of the upland drainage), the primary input to the lakes is from Lacamas Creek and its tributaries. The lakes are popular recreational facilities for brown trout fishing, swimming, boating, and water skiing.

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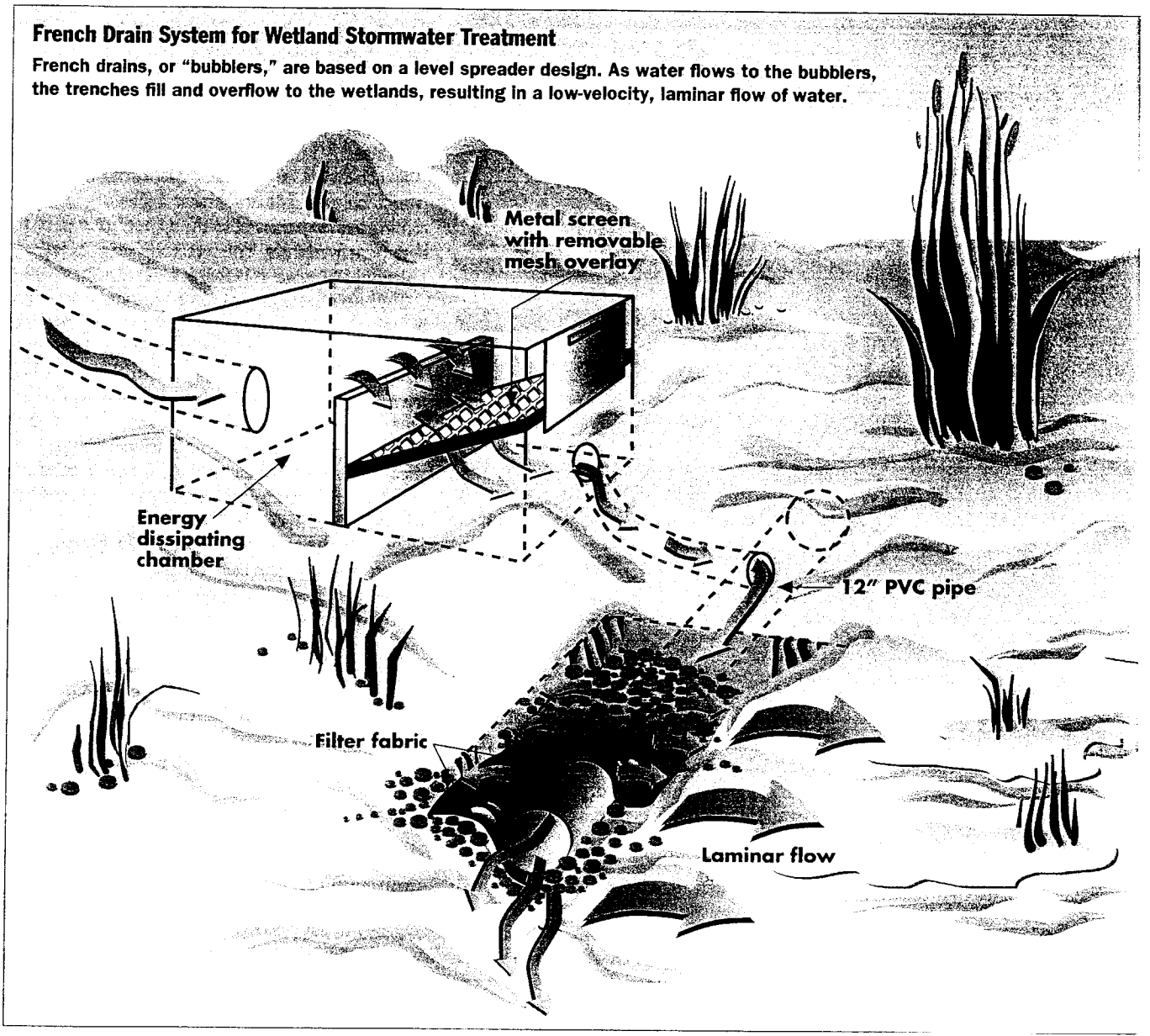

French Drain System for Wetland Stormwater Treatment

French drains, or "bubblers," are based on a level spreader design. As water flows to the bubblers, the trenches fill and overflow to the wetlands, resulting in a low-velocity, laminar flow of water.

EXISTING WATER QUALITY ISSUES

Because of the existing water quality in the Lacamas Lake watershed, there was some concern about the additional loading from stormwater runoff that the development would bring. An EPA Phase I diagnostic and restoration study showed that both Lacamas Lake and Round Lake suffer from severe overenrichment by nutrients, particularly from agricultural inputs and failing septic systems upstream of the lakes. Of these two sources, animal waste from the large number of dairy and animal-producing farms in the watershed is the primary contributor. Pollutants associated with septic failure have been increasing as residential development expands throughout the watershed.

Nearly 96% of the annual phosphorus loading to Lacamas Lake is from watershed sources, leaving a relatively minor contribution from precipitation, groundwater, or in-lake sources (for example, macrophytes and sediments). Lacamas Lake and Round Lake are classified as eutrophic to hypereutrophic based on this level of enrichment.

The resultant water quality problems in the lakes include severe dissolved oxygen depletion, poor water clarity, and extensive algal growth, particularly during the high-use summer recreational months.

At present, there are no limits for total phosphorus for the watershed area. The Clark County Water Quality Division has collected water quality data in the Lacamas Creek watershed in anticipation of such limits, but has not begun this type of analysis.

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Compliance Criteria Determined from Site Monitoring

Parameter	Units	Compliance level[a] Bubbler, wetland 1 (S4)	Compliance level[a] Bubbler, wetland 2 (S2)
Primary			
[illegible]	[illegible]	[illegible]	[illegible]
PO_4	mg/L	0.061 [0.131]	0.042 [0.090]
[illegible]	[illegible]	[illegible]	[illegible]
Secondary			
[illegible]	[illegible]	[illegible]	[illegible]
Conductivity	µmho/cm	63 [97]	87 [135]
[illegible]	[illegible]	[illegible]	[illegible]
Oil and grease	mg/L	1.5 [4.2]	1.8 [4.5]
[illegible]	[illegible]	[illegible]	[illegible]
Copper	mg/L	Wash. wq stds.	Wash. wq stds.
[illegible]	[illegible]	[illegible]	[illegible]
Zinc	mg/L	Wash. wq stds.	Wash. wq stds.
[illegible]	[illegible]	[illegible]	[illegible]
Chlorinated pesticides	µg/L	detection limit	detection limit
[illegible]	[illegible]	[illegible]	[illegible]

[a]Numbers in brackets are the compliance concentration plus twice the standard deviation.
[b]Wash. wq stds. = Washington state water quality standards.

Water quality data collected during the Lacamas Lake-Round Lake study suggested that phosphorus is the limiting nutrient controlling the amount of plant growth in the lakes. Thus, it was important that developers of the residential units control phosphorus loading in stormwater runoff.

FRENCH DRAINS DIRECT RUNOFF BELOW GRADE

The developers considered both detention ponds and grassy swales as treatment options; however, site constraints (including proximity to the lakeshore and existing forested wetlands) and the desire not to excavate adjacent to the lake eliminated the use of detention ponds. The use of a wetland biofilter resulted from an attempt to mimic the laminar flow of runoff that existed before development. In addition, the area above existing forested wetlands could be used to create additional wetlands to treat and detain the runoff. At the time of the design of the project, using wetlands for stormwater treatment was considered a beneficial use, on the same level as providing habitat, groundwater recharge, or recreation. In addition, wetlands were not considered waters of the state under the federal Clean Water Act and could therefore be used for stormwater discharges. However, the Washington Department of Ecology now requires that stormwater be pretreated before discharge to waters of the state, including wetlands.

The Lacamas Lake wetlands biofilter was designed to release the collected stormwater to the wetland area at noneroding velocities [< 0.304 m/s (1 ft/s)]. Peak flows for storm events were estimated for the development area using the HEC-1 model, developed by the U.S. Army Corps of Engineers. (HEC stands for Hydrologic Engineering Center.) The distribution pipe system was then sized to release water at the required velocities.

French drains, or "bubblers," were designed to direct runoff below grade and create a sheet flow several centimeters deep that enters the upgradient edge of the emergent wetlands. The wetlands then "treat" the inflowing stormwater before it enters Lacamas Lake.

The bubbler design consists of a section of 0.3-m (12-in.) pipe embedded in 0.05 m (2 in.) of gravel (see Figure, *French Drain System for Wetland Stormwater Treatment*). The pipe has 0.01-m (0.5-in.) holes that have been drilled at 0.1-m (0.5-ft) intervals at mid-depth of the pipe [0.1 m (0.5 ft) above the pipe invert along both sides of the pipe]. The gravel-filled trench perpendicular to the slope is lined with filter cloth. The perforated pipe is placed in the trench, covered with filter cloth, and then the trench is back-filled with gravel (see Figure). Sediment traps were installed upstream from the bubblers to prevent them from clogging.

As water flows to the bubblers, the trenches fill and overflow to the wetlands, resulting in a low-velocity, laminar flow. The finish grade over the top of the bubbler pipes must be level to prevent channelization caused by uneven flow. Field observations have confirmed that the bubblers were properly installed; during storm events, sheet flow can be observed just below the bubblers.

MONITORING PLAN USED TO DEVELOP CRITERIA

The Washington Department of Ecology would not allow the use of a wetland system for stormwater treatment until compliance levels were established for nutrients in runoff leaving the site. In the absence of any state or local watershed loading limits, self-imposed water quality criteria were developed based on data collected at the site. A 5-year monitoring plan was implemented by the developer with input from the state Department of Ecology and the city of Camas. Data sets from the first 2 years of monitoring were used to develop the criteria (see Table, *Compliance Criteria Determined from Site Monitoring*). A total of 11 complete sampling periods provided water quality data from the wetland's surface water and groundwater, Lacamas Lake, and control creeks (creeks not receiving runoff from the development). Additional information from water-table measurements, observations on vegetation health and composition, and determination of pesticide, herbicide, and heavy metal levels was gathered regularly during the same period.

The monitoring plan was designed to obtain baseline water quality data for surface water and groundwater entering and exiting the wetlands, determine the effectiveness of the wetland system as a biofilter for

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stormwater runoff, and establish criteria for implementing contingency plans if water quality data indicated unacceptable levels.

During monitoring, stormwater was collected and distributed to the wetland so that the parameters of concern could be monitored at specific points as the water flowed into and out of the wetlands. These points were sampled during storm events to provide representative measurements of water quality before and after application to the wetland.

Transects were established to monitor water quality through the wetlands. As water flows from the bubblers through the wetland, the distance downstream from the bubblers is directly related to the length of time that the water was retained in the wetland.

Sites were selected along the two transects for monitoring groundwater levels, surface water and groundwater quality, and vegetative cover and composition (see Map for reference points). Water quality sampling points included the sediment traps above the wetlands (B1, B2), surface water flow into and out of the wetlands (S2, S3, S4), shallow groundwater wells (G1 through G7, plus G11), and two lake sites (L1, L2) just outside the mixing zone of each input channel from the wetlands. Transect 1 included B1 through S4 and terminated at L2. Groundwater in transect 1 was collected in G4 through G7 and G11. Transect 2 ran from B2 to S2, terminated at L1, and included groundwater wells G1 through G3.

To provide background information on the quality and quantity of surface water runoff, three control creeks were also sampled. Creek 1 (C1, Dwyer Creek) empties into lower Lacamas Creek above the lake, north of the development. Two unnamed creeks on the lake's eastern shore (C2 and C3) were also sampled and represent contributions from smaller, relatively undeveloped watersheds.

Sample dates included a range of high and low flows, the "first flush" episode following the summer dry season, and an early-winter sampling after the fall first flush to detect the presence and extent of any decrease in water quality levels. During each of the sample periods, the wetland areas between the bubblers and the outflow streams (S2 and S4) were inspected for channelization.

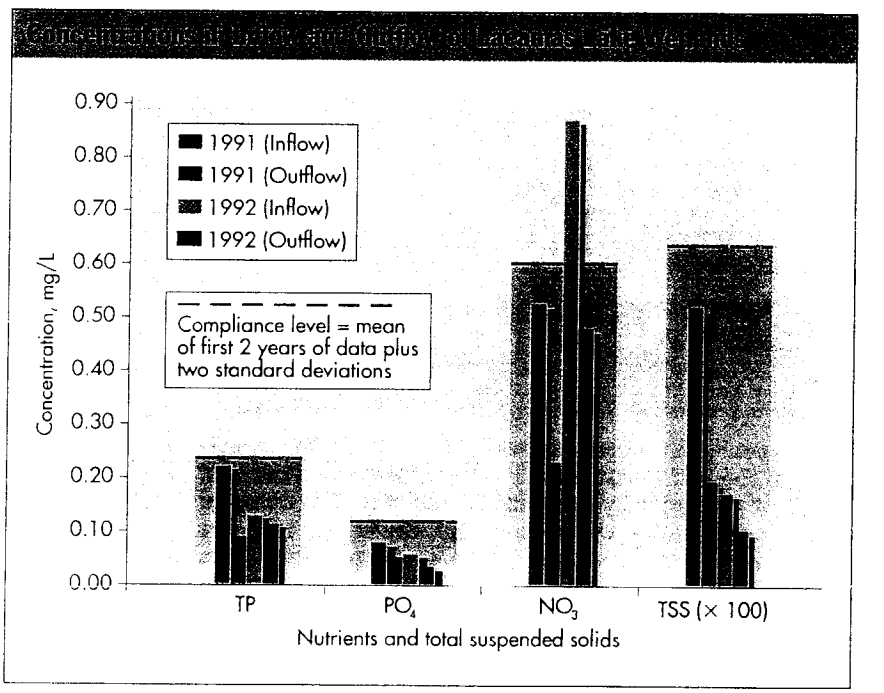
Sampling of storm events consisted of routine field measurements for temperature, conductivity, pH, and groundwater depths. Analyses of nutrient concentrations [total phosphorus, soluble phosphorus, and nitrate-nitrite] and total suspended solids were also performed.

Additionally, samples from selected stations were analyzed during the first heavy rain of the fall, when accumulated sediment and contaminants are washed from roads and land surfaces. Levels of oil and grease, organophosphate pesticides, chlorinated herbicides, organochlorine pesticides, and dissolved metals (chromium, copper, lead, and zinc) were measured along with the other routine parameters.

A set of site-specific criteria derived from baseline water quality data was proposed to define acceptable wetland performance and vegetative change resulting from stormwater treatment. Water quality values more than two standard deviations from these baseline conditions were considered suspect and warranted further investigation into their origin and impact.

COMPLIANCE WITH SITE-SPECIFIC CRITERIA

The most important criteria were related to nutrients and vegetation changes. Additional threshold criteria for pesticides, herbicides, metals, and biota (plant health and longevity) were used to evaluate the quality of the stormwater that passed through the wetlands before entering the lake during the subsequent 3 years of monitoring.



In the third and fourth years (1991 and 1992), a trend between the average annual inflow and outflow concentrations was apparent. Primary parameters (TP, PO_4, and NO_3/NO_2) and secondary TSS measured at outflows were less than inflow concentrations, indicating that the wetlands were removing nutrients and solids from stormwater runoff (see Figure, *Concentrations in Inflow and Outflow of Lacamas Lake Wetlands*).

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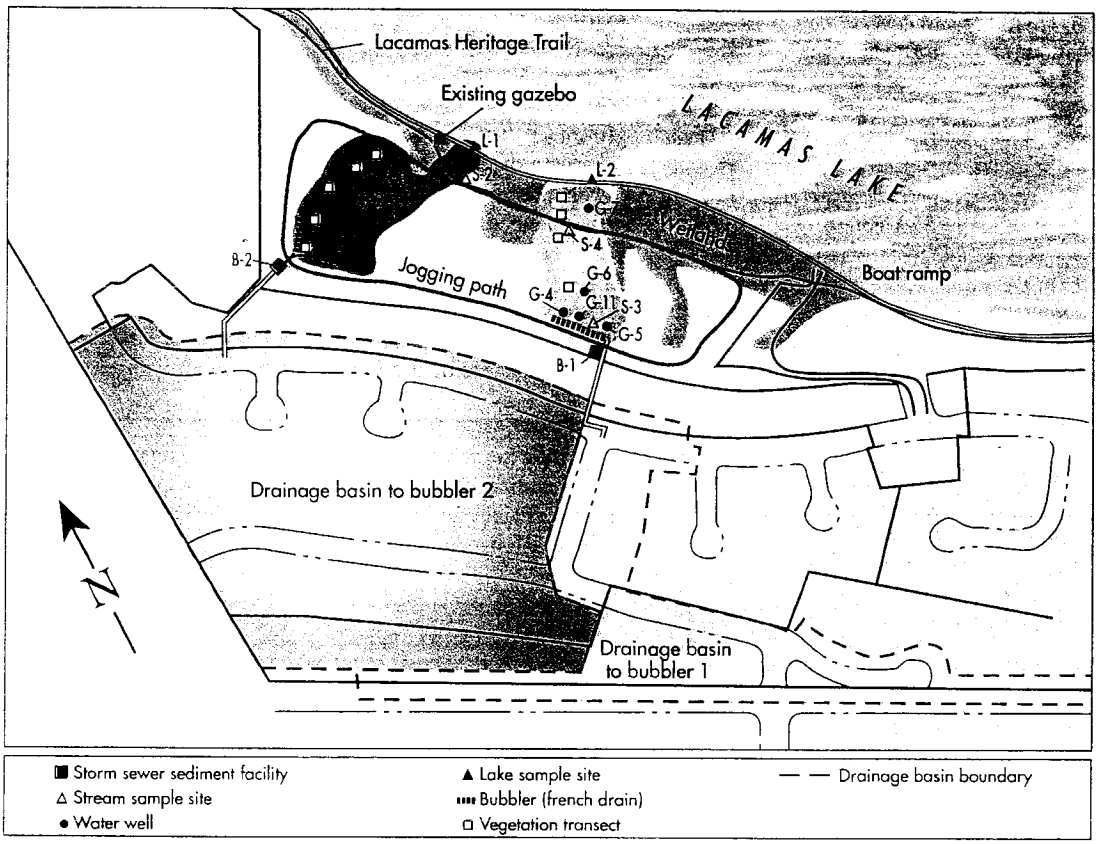

Wetland Stormwater Treatment Site Description

The residential development is located on the southwestern shore of Lacamas Lake. The southern two-thirds of the site is occupied by residential home lots. The northern third is composed of wetlands and grassy areas. The area directly adjacent to the lake is a forested wetland 15.2 to 30.4 m (50 to 100 ft) wide. From this forested area, areas of emergent wetlands extend southeast 121.6 to 182.4 m (400 to 600 ft).

The development site is steep. Surface elevation drops from 106.4 m (350 ft) at the south edge of the project to 57.8 m (190 ft) at the lakeshore. Before development, the area had five major surface-drainage features. As the site has been developed, stormwater runoff from paved surfaces and the flow in four streams has been collected by pipes and catch basins and discharged into the French drains ("bubblers") located along the upgradient edge of the wetlands. The four northern streams are spring-fed and usually flow during dry weather. The fifth stream adjacent to the road and the boat ramp is intermittent and flows only during periods of significant rainfall.

• • • • • • • • • • • • •

Total phosphorus in surface water flowing out of the wetlands (the limiting nutrient in the lake) was consistently below established compliance levels.

No herbicides or pesticides were detected in the first flush samples for 1991 and 1992. Dissolved metals (chromium and zinc) were detected in selected samples, but at levels equal to or less than EPA drinking water standards.

Compliance levels, excluding TP, were exceeded on two of the six monitoring dates during the third year of monitoring. The nitrate concentration at station S4 (below the bubbler in wetland 1) was slightly above the upper compliance level. The soluble phosphorus and nitrate criteria were also exceeded at S2 and S4, respectively. These two parameters were in compliance for the remainder of the monitoring season. Increased nitrate values during the winter warranted closer scrutiny during the fourth year of monitoring to see whether the trend continued.

During the fourth year, the compliance level for nitrate was exceeded on two of the four monitoring dates. As in the third monitoring year, the levels were exceeded in the winter sampling months, and nitrate values for the two transects were in compliance for the remainder of the monitoring season.

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Phosphorus Loading Rates and Annual Phosphorus Load

Station	Drainage basin areas, ha[a]	1990 Loading rate, kg[b]/ha·yr	1990 Phosphorus load, kg	1991 Loading rate, kg/ha·yr	1991 Phosphorus load, kg	1992 Loading rate, kg/ha·yr	1992 Phosphorus load, kg
Bubbler, wetland 1 (S4)	4.1	0.702	2.9	0.724	3.0	0.690	2.8
Bubbler, wetland 2 (S2)	8.8	0.872	7.7	0.543	4.5	0.716	6.3
Dwyer Creek (C1)	324	1.002	325	1.373	445	1.359	440
Unnamed Creek (C2)	46.5	1.099	51	0.868	40	0.914	43
Unnamed Creek (C3)	66.8	1.292	86	1.131	76	1.222	82

[a]hectares x 2.471 = acres.

[b]kilograms x 2.205 = pounds.

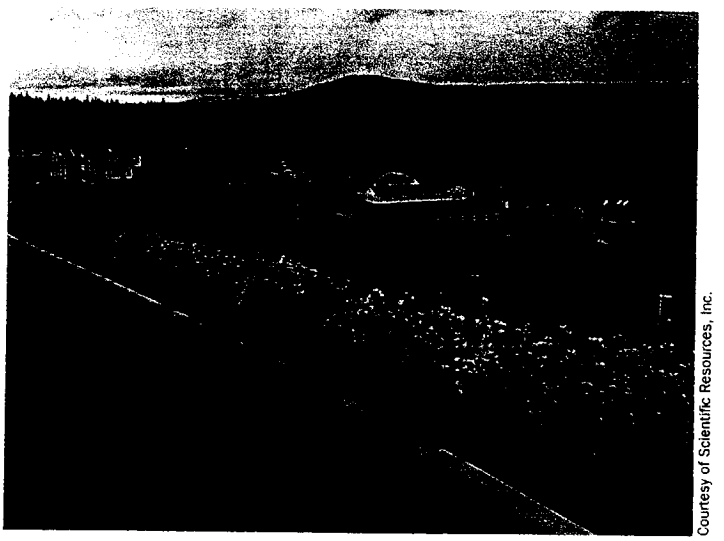
ESTIMATING PHOSPHORUS LOADING

The impact on lake water quality from stormwater passing through the wetlands is only partially determined by comparing the compliance parameters of interest to the site-specific criteria. Determining the nutrient load contributed by various input streams can define the actual quantity of nutrients going to Lacamas Lake. Using TP as an indicator of nutrient loading, calculations of the annual phosphorus contributions per unit area for the second, third, and fourth monitoring years were made for the development and compared with inputs calculated from three control creeks (see Table, *Phosphorus Loading Rates and Annual Phosphorus Load*). In all cases, nutrient loading from the development is lower than from the three control creeks for these 3 years.

It is not possible to compare the actual phosphorus loading rates that were estimated for the development with established limits for the Lacamas Creek watershed because no regulatory framework for phosphorus loading limits exists. Moreover, there were no reports of other developments, residences, or businesses in the watershed currently monitoring loading to the lake. Data from the water quality sampling data collected will hopefully allow the county to begin developing loading limits for the Lacamas Lake watershed.

EVALUATING PERFORMANCE

One of the original concerns of using the wetlands for stormwater treatment was the ability of the system to reduce phosphorus loading to Lacamas Lake. All evidence to date indicates that the bubbler systems are reducing concentrations of phosphorus from the development to levels below site-specific criteria. An unanticipated consequence of sending stormwater to the treatment wetland has been the increased input of nitrate, with the site-specific criteria often exceeded during the winter months. However, the increased nitrate input is not in violation of state or local water quality standards and is occurring at a time when the lake is well flushed and there is reduced algal and plant growth. A diagnostic and restoration study also indicated that the lake was a phosphorus-limited system during the winter, so that added nitrate would not adversely affect the trophic level of the lake. In the absence of watershed loading limits for either nitrate or phosphorus, it is not possible to determine whether the winter nitrate contribution from the development is significant to the lake nutrient budget as a whole.



Courtesy of Scientific Resources, Inc.

Lacamas Lake's stormwater wetland treatment system has reduced nutrient and sediment loading to the lake.

Future compliance with water quality criteria may entail alterations of the wetlands to optimize treatment. These alterations would increase the area of wetlands or increase the detention time of stormwater within the wetlands. Another approach to ensuring future compliance would be to limit the sources of nutrients flowing into the wetlands. A source control program including proper lawn and garden practices would provide opportunities for reducing the amount of nutrients in stormwater runoff and would be a vital component in the success and longevity of the wetland treatment system. ■

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R0021596

Promoting Stormwater Education

Educating the regulated community is the key to the success of the EPA stormwater program

Rhonda E. Harris
Paulette Johnsey
Brent Larsen
Monica Berrell

Educating and involving the public in any new regulatory program is among the most important factors responsible for a program's success. The EPA stormwater program, under the National Pollutant Discharge Elimination System (NPDES), has recognized the value of public education, from the original development of the regulations to the public hearing process and the implementation of the regulations. (For additional information on the NPDES stormwater program, see "Stormwater's October Deadline Looming," in the September 1992 issue of *WE&T*.) Issues such as regulatory deadlines and implementation procedures all depend on educating both the regulated community and the public at large.

EPA is now faced with the challenge of implementing one of its largest programs to date. The regulations requiring permit application for stormwater discharges bring together a distinctly different community in the sense that these discharges are dependent on the whims of Mother Nature. The regulated community for the stormwater

Courtesy of the City of Fort Worth Environmental Management Division

With proper training, program participants can use sampling kits to detect spills or illicit connections affecting stormwater.

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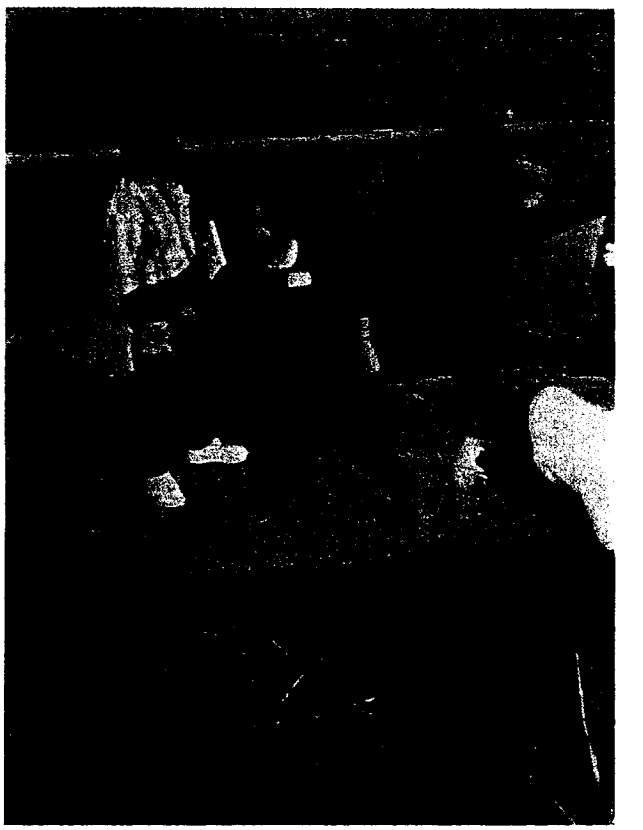
program includes all large- and medium-sized cities and unincorporated urban areas with a population of 100,000 or more, and a very large, diverse group of industries. Each of these entities is required to develop a stormwater management plan that includes some public or employee education. There are many layers of education in each of the areas affected by the regulations, including

- education within the industries covered by the stormwater regulations,
- education within the cities covered by the stormwater regulations, and
- education provided to other federal agencies about the stormwater regulations and the permitting processes.

The stormwater program is not likely to succeed without a strong education program that operates at both the internal and external levels in each of these categories (industries, cities, and federal agencies). For example, each of these entities is likely to be involved in education activities for external customers and the public as well as their own internal employee education programs.



EDUCATION IN INDUSTRIES

Industries covered by the stormwater regulations will be required to implement educational programs in several areas. Many industries already have employee training programs in place and will be able to use these to educate their employees concerning stormwater regulations. Other spill prevention programs, contingency procedures, and response procedures already in place in a particular industry may be adapted to situations dealing with stormwater discharges. Industries without such mechanisms will be required to develop them.

Regardless of the current status of an industry training program, it is critical that industries develop a stormwater pollution prevention plan with an emphasis on employee education and training. As employees become familiar with the pollution prevention plan, they can begin to implement it throughout the workplace. The better trained employees are, the more successful the stormwater management program will be. In time, training will benefit both the industry and the community as a whole.

Under the program, all employees need to be informed of the components and goals of their employer's stormwater pollution prevention plan, which must include:

- a *pollution prevention committee* to be responsible for developing the stormwater pollution prevention plan and assisting the plant manager in implementation;
- a *risk assessment/material inventory* to assess the potential of various sources to contribute pollutants to stormwater discharges, including an inventory of the types of materials handled;
- a *preventive maintenance program* to inspect and maintain all stormwater management devices and inspect and test plant equipment and systems to uncover conditions that could cause failures that would result in pollutant discharges in the stormwater;
- a *good housekeeping program* to require the maintenance of a clean, orderly facility;
- *spill prevention and response procedures* to identify potential spill areas, drainage points, material-handling procedures and storage requirements, cleanup procedures, and necessary equipment available to personnel;
- a *stormwater management program* that includes a narrative summary of the appropriateness of traditional stormwater management practices, based on an assessment of the potential to contribute pollutants to stormwater discharges, with reasonable and appropriate measures to be implemented and maintained;
- a *sediment and erosion control plan* to identify areas with a high potential for significant soil erosion and identify measures to limit such erosion;
- *employee training programs* to inform all personnel of the components and goals of the stormwater pollution prevention plan, with periodic dates identified for training;
- *visual inspections* of plant equipment and areas to identify evidence of, or the potential for, pollutants entering the drainage system, with tracking and followup procedures; and
- *record-keeping and internal reporting procedures* to document spills, discharges, and information describ-

R0021599

Courtesy of the City of Fort Worth Environmental Management Division

Stormwater education programs should include training in safe sample collection procedures.

ing the quality and quantity of stormwater discharges, inspections, and maintenance activities.

Adequate tracking, record-keeping, and internal reporting systems, in particular, will require extensive employee education to set up and implement on a day-to-day basis.

EDUCATION IN CITIES

At present, large- and medium-sized cities covered by the stormwater regulations are required to develop and implement a stormwater management plan as a part of their applications for a *municipal separate storm sewer system* (MS4) permit. The management plan includes a comprehensive planning process that will involve public participation. The process, which includes a description of the staff and equipment available to implement the program, may be imposed on a systemwide, watershed, or jurisdictional basis, or on individual outfalls. In addition, where necessary, there will also be intergovernmental coordination to reduce the discharge of pollutants from the system to the maximum extent practicable, as well as control techniques, system design and engineering methods, and other provisions as appropriate.

A key element of the management plan is public participation, which must include extensive public education activities to assist communities in understanding the importance of this program and their impact on it. Specific education activities can include

- information flyers included in utility bills sent to residents;
- direct public contact, including open houses at specific facilities, school education programs, or a speaker's bureau for schools and community groups;
- partnerships with civic organizations (for example, Boy Scouts and Girl Scouts);
- citizen advisory groups; and
- cleanup campaigns.

One of the most important areas in which cities can focus their education efforts is in educating the public about *household hazardous wastes*. Cities need to include development of a household hazardous waste pickup program as a part of their management plans. Along with this program, cities can develop a program that defines household hazardous wastes, explains how they should be handled and stored, and outlines disposal requirements.

The North Central Texas Council of Governments is currently developing a household hazardous waste disposal program to serve its 16 member counties that is based on a successful program in place in King County, Wash. The program includes recruiting and training volunteers to educate a variety of neighborhood and civic groups about household hazardous waste pollution prevention. The council also supports the public relations work of local Keep Texas Beautiful/Keep America Beautiful affiliates and city recycling coordinators. This concept can be expanded further and shared with other metropolitan areas to promote public education.

Historically, public works departments of municipalities have been concerned only with the quantity of stormwater and how to remove it from the streets as quickly as possible. Concern about the *quality* of stormwater is a new concept to these employees. Because municipalities must screen their outfalls in both dry weather and wet weather under this new program, employees need to be trained in new techniques for inspecting and testing stormwater samples.

Safety training associated with the discharge screenings is also essential in both dry weather and wet weather. Most often, outfalls are located in areas that are dangerous in terms of access and sampling. Steep slopes, slippery conditions, poisonous plants, snakes, insects, and small animals make getting a good sample from a stormwater outfall very challenging. Municipalities must take every precaution for their employees, including training them to deal with adverse conditions.

A benefit of a city public education campaign for the stormwater program is troubleshooting and problem reporting by the public. To take full advantage of the public's watchfulness in dry-weather screening pro-

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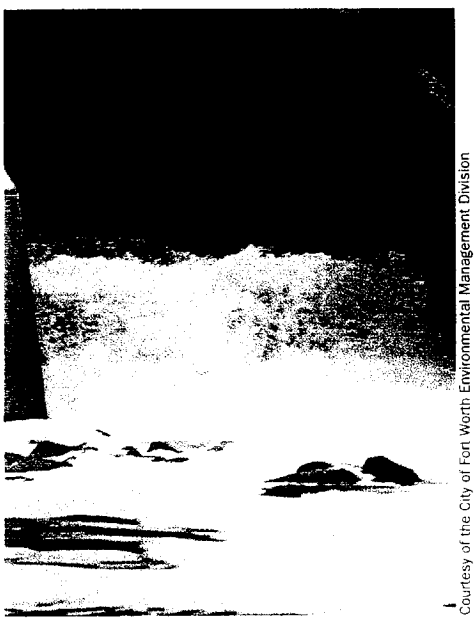
grams, cities can develop reporting criteria and procedures for the public to follow. The information needs to be clearly stated, public participation should be voluntary, and the city should not be liable if someone is injured in attempting to collect information.

These reporting procedures are similar to crimewatch or fraud-reporting programs and could even include a hot line for the public to report dry-weather discharges. The public participation program could possibly reduce the labor required to perform a large-scale dry-weather screening program, or at least locate more discharges than could be done by staff alone. Dischargers would be consistently reminded that the public is watching and has access to the system, encouraging compliance with the city's management plan.

Municipalities covered by these regulations in Texas also have the Clean Texas 2000 campaign, which was developed to form a partnership with businesses, local governments, communities, organizations, and cities to reduce and prevent pollution; to increase environmental awareness among all Texans and achieve the long-term behavior changes necessary to ensure a cleaner and healthier environment; and to facilitate environmental protection throughout the state beyond federal and state requirements. The goal of Clean Texas 2000 is to reduce the release of toxics and the generation of hazardous pollutants in Texas by 50% or more overall from 1987 levels by 2000.



Courtesy of the City of Fort Worth Environmental Management Division

EPA protocol requires that storm events be documented.

A major component of this campaign is public education to increase environmental awareness among Texans and to increase their participation in pollution control. The benefit of public education has been felt most when municipalities determine the finances needed to implement the stormwater program. Some cities have embarked on an aggressive education campaign to let the people know what they will be getting in exchange for the new fees. While there has been some resistance because of the country's recent economic setbacks, the public understands what cities are trying to accomplish through these programs.

Cities that have tried to push a stormwater program through without adequate publicity have often encountered resistance by the public. Citizens often contact officials complaining about the "rain tax," and it is difficult to get agreement on a new fee or tax without telling the public what it will be used for.

Even with public education, implementation is not always successful. In one Texas city, the mayor and city council approved a stormwater utility and set up a fee structure to provide funds for its application. In the next city election, every council member who had voted for the utility, and the mayor, were voted out of office and the utility was overturned. As a result, the city has struggled to meet its statutory deadlines.

EDUCATION AND FEDERAL AGENCIES

EPA has set up a National Stormwater Hot Line to provide information to callers about the stormwater regulations and their implementation. By necessity, however, many of the actual permitting decisions must be made at the regional level. Therefore, EPA Region VI has set up a regional hot line to answer many of these questions. The regional hot line receives as many as 100 calls daily from concerned applicants and citizens wishing to know more about how these new regulations will affect them.

EPA Region VI provides speakers for seminars, special engagements, and training sessions to try to reach as many people as possible. In general, these efforts have been successful. However, the regulations cover a broad spectrum of industrial activities, and only limited resources for outreach activities are available.

EPA is also concerned with the education of other persons and agencies within the federal government so that they may understand how the stormwater program works within or through their own programs. This is particularly true with the U.S. Army Corps of Engineers and the Soil Conservation Service, which deal with specific aspects of stormwater discharge and runoff in their programs. These agencies must be aware of how their decisions will affect the quality of stormwater and how that quality will, in turn, affect U.S. waters.

These education efforts continue to be an important aspect of the stormwater program as more people become aware of the regulations and how they affect their daily lives. Industries will begin to develop their own education programs for employees, customers, and the public; cities will begin to train their employees in sampling techniques and safety procedures, as well as develop public information programs; and EPA will continue to participate in outreach to all areas affected by the stormwater regulations, both internal and external. ■

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R0021601

Surviving the Stormwater Permit Process

Permit options and monitoring methods under the NPDES process

Bob Bailey ■

Since EPA published regulations for stormwater discharge permits, industrial facilities and municipalities have had to consider new and timely methods for compliance. In essence, the regulations require both industries and municipalities to develop stormwater controls similar to those currently required for point source discharges, solids, and process wastewater discharges. National Pollutant Discharge Elimination System (NPDES) permit applicants must submit a detailed stormwater pollution prevention plan.

NPDES PERMIT OPTIONS

When applying for NPDES stormwater permits, dischargers have three options:

- individual permit applications,
- general permit applications (also called "notices of intent"), and
- group permits.

Individual permits. Individual permits are issued to a specific industrial facility. The permit is generally tailored to the discharge characteristics of the permit holder or to the special requirements of the receiving waters. The deadline for applying for individual permits was October 1, 1992. The permit application consists of two parts. The first, Form 1, includes such basic information as name, address, standard industrial classification (SIC code), and existing environmental permit numbers. The second part, Form 2F, is composed of a site drainage map, a narrative description of the site identifying potential pollution sources, and quantitative sampling data taken during a storm event.

Most applicants are directed by administrative authority to apply for a general permit first and request an individual permit later. While the NPDES individual permit is the most costly option, many believe it offers a greater degree of control because the applicant can negotiate discharge limits with the permit writer. However, from EPA's point of view, the starting point for negotiations is the general permit.

General permits. General permits cover most industrial discharges throughout a state, as long as the state has the authority to issue general permits.

R0021602

Oil and grease and total suspended solids in stormwater can have serious impacts on receiving waters.

Currently, 38 states have such authority, with compliance deadlines and conditions set by the state. In states without EPA-delegated authority to issue general permits, EPA's core general permit applies. Confusion about whether the Office of Management and Budget had approved EPA's original general permit draft caused some municipalities and industries to apply for individual or group NPDES permits to meet the initial 1991 application filing deadlines. These deadlines were subsequently extended to October 1, 1992.

The general permit might be perceived as the easiest and least costly of the three compliance methods because it requires only routine stormwater sampling in some cases and no sampling in others. However, there are severe criminal penalties for violating permit conditions; for example, "Section 309(c)(4) of the Clean Water Act provides that any person who knowingly makes any false material statement, representation, or certification in any record or other document submitted or required to be maintained under this (NPDES) permit, including reports of compliance or noncompliance shall, upon conviction, be punished by a fine of not more than $10,000, or by imprisonment for not more than 2 years, or both." The Clean Water Act also provides that "any person who falsifies, tampers with, or knowingly renders inaccurate any monitoring device or method required to be maintained under this permit shall, upon conviction, be punished by fines and imprisonment described in Section 309 of the CWA."

If sampling is not required, the permit holder is required to include the certified statement that "material handling equipment or activities, raw materials, intermediate products, final products, waste materials, by-products, industrial machinery or operations, significant materials from past industrial activity, or, in the case of airports, deicing activities, that are located in areas of the facility that are within the drainage area of the outfall are not presently exposed to stormwater and will not be exposed to stormwater for the certification period."

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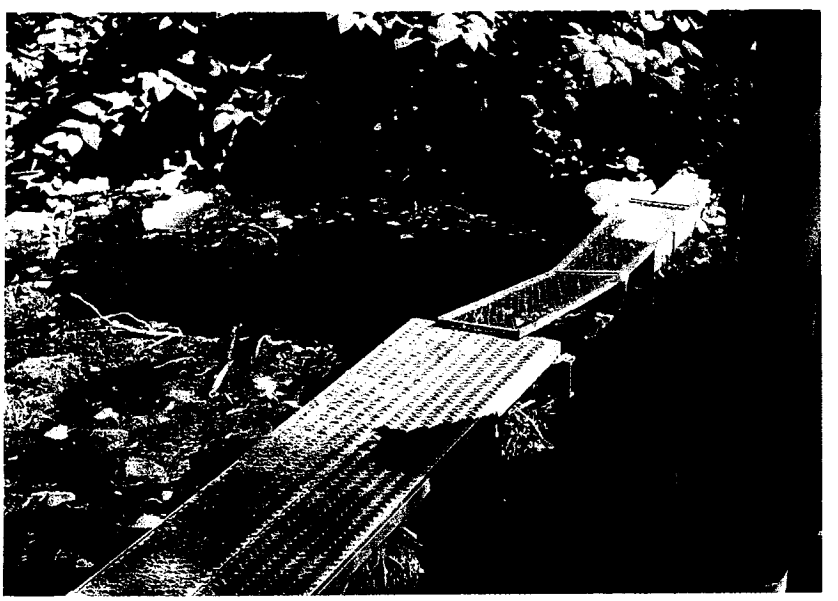
Stormwater Defined

EPA defines stormwater as "stormwater runoff, snow melt runoff, and surface runoff and drainage." Stormwater associated with industrial activity is further defined as "the discharge from any conveyance which is used for collecting and conveying stormwater and which is directly related to manufacturing, processing, or raw materials storage areas at an industrial plant." Several recent EPA studies have shown that runoff from urban and industrial areas typically possesses pollutant concentrations rivaling those of industrial wastewater discharges; as much as 30% of water pollution streams can come from stormwater discharges.



Stormwater permit applications must include a map that shows topography, drainage and discharge structures, and paved areas and buildings.

The deadline for submitting a notice of intent to be covered by an EPA-administered general permit was October 1, 1992, although the EPA has been lenient about late permit applications. Facilities that began industrial activity after October 1, 1992, are required to submit a notice of intent at least 2 days before the beginning of industrial activity at the facility.

Group permits. Group permits cover dischargers within a particular industrial group or with similar discharge characteristics. A group may comprise as few as four entities; the process is intended to reduce the expense and administrative burden on both industry and the permitting authority by requiring only selected members of the group to submit quantitative data. The group permit has a two-part filing process. Part 1 identifies all participants, provides facility-specific information, and proposes a representative sampling subgroup. Part 2 consists of sampling data from each member of the subgroup identified in Part 1.

The group permit is a subset of the general permit track and is designed to generate customized general permits for those within the group, as long as the permitting agency accepts everyone in the group. The possibility of having to apply for an individual permit still exists.

Filing deadlines for municipalities owning or operating facilities with industrial activity and possessing separate storm sewers follow:

- The Part 1 deadline for large municipalities (populations exceeding 250,000) was November 18, 1991, while for medium municipalities (from 100,000 to 250,000) the deadline was May 18, 1992.
- The Part 2 deadline for large municipalities was November 16, 1992, while for medium municipalities it was May 17, 1993.

For Part 2 of the NPDES permit application, municipalities must submit monitoring data for 3 storm events gathered from 5 to 10 outfalls.

TESTING FOR BASIC POLLUTANTS

Stormwater sampling for NPDES permits covers eight pollutants and parameters, including oil and grease, 5-day biochemical oxygen demand (BOD_5), chemical oxygen demand (COD), total suspended solids (TSS), nitrate and nitrite-nitrogen, total Kjeldahl nitrogen (TKN), total phosphorus, and pH.

Oil and grease and TSS are common components of stormwater and can have serious impacts on receiving waters. Measuring oxygen demand (BOD_5 and COD) assists in evaluating the oxygen depletion potential of the discharge. The pH provides the permitting authority with information on the potential availability of metals to receiving flora, fauna, and sediment. Nitrate and nitrite-nitrogen, TKN, and total phosphorous nutrient measurements indicate the affects of stormwater on water quality. Additional required testing of stormwater samples for industry-specific pollutants and parameters is included in the core general permit.

MONITORING METHODOLOGY

While current regulations governing permit applications call for monitoring one to three representative storm events (depending on whether the applicant is industrial or municipal), in some EPA-specified industries, annual or semiannual sampling is mandatory. In addition, stormwater pollution prevention plans for EPA general permits were required to be prepared by April 1, 1993, implemented by October 1, 1993, routinely evaluated, and documented.

EPA protocol requires that

- sampling begin after 2.54 mm (0.1 in.) of rainfall, with 72 hours of dry time having elapsed since the last 2.54-mm (0.1-in.) storm event;
- a first-flush sample be taken within the first 30 min-

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utes of the onset of the storm event;

- a flow-weighted composite sample be taken at the rate of one every 15 minutes, covering the duration of the storm or 3 hours, whichever is less; and
- the storm event be within 50% of the average or median duration and amount of rainfall for that area. This requires monitoring of rainfall from the beginning to the end of the storm to determine whether it meets the 50% criteria.

First-flush samples often consist of larger-than-normal quantities of unattached fine particulates and soluble surface material that have accumulated between storms. Thus, the first moments of a storm contribute the highest concentration of pollutants to the receiving stream. Therefore, EPA requires that a grab sample be taken during the first 30 minutes and analyzed separately from subsequent "duration" samples. Levels of total pollutants are measured by combining the flow-weighted composite sample and the volume discharge to determine pollutant loading to the receiving stream.

MANAGING THE SAMPLING PROCESS

The data-gathering requirements for NPDES permits involve a specific sampling protocol over an unpredictable period in potentially hazardous conditions. The unpredictability of the weather means that individuals have to be on call and ready to respond. If the site has been preequipped with a recording rain gauge, personnel have at least 30 minutes to get to the site and collect the first-flush samples. If there is no recording rain gauge, personnel must be at the site before the rain starts and remain for the entire event to determine the storm's duration and total rainfall.

Automatic equipment decreases the staff required and eliminates the quick response necessary. In addition, workers do not have to remain at the site from the time the rain begins until the storm ends. While samples are required for only 3 hours, measurement of rainfall and flow must continue for the duration of the storm.

Safety. Manual sampling exposes personnel to the elements and potential hazards such as lightning, inaccessible monitoring sites, steep embankments, and slippery footing. Storm conveyances also often have powerful flows. For example, a flow with a velocity of 0.75 m/s (2.5 ft/s) and a depth of 0.5 m (1.5 ft) can cause a full-grown man to be swept away. Storm sewers can have velocities of 6 m/s (20 ft/s) and depths of 3 m (10 ft) or more. The flows can be sudden and are dangerous.

Automatic equipment eliminates the need to expose personnel to the elements and associated hazards. It can be installed at an outfall and then programmed in advance of any storms.

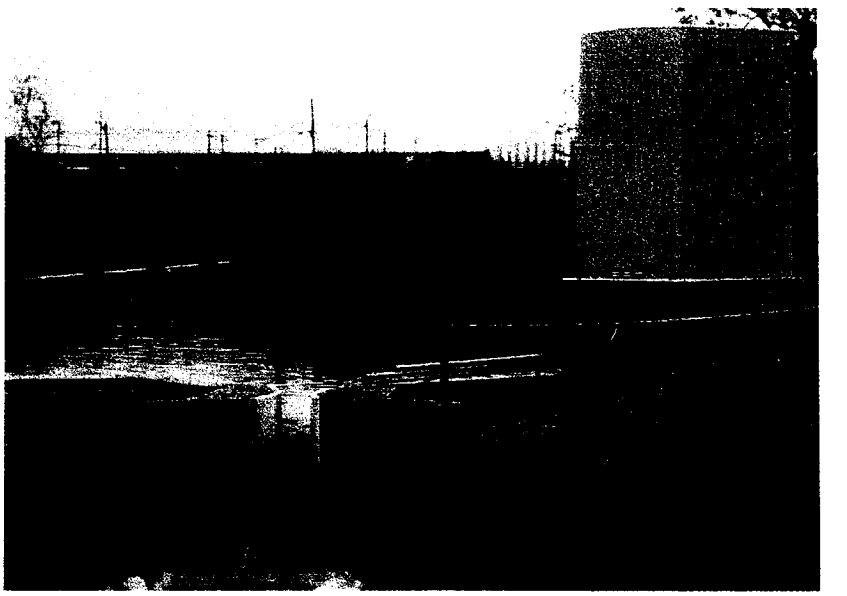
Documenting the storm event. Stormwater regulations require that information be recorded before, during, and after the storm event. This information can be written down manually or logged by automatic sampling equipment in electronic memory. However, field documents are often damaged by exposure to rain.

Data quality. Manual collection is repeatable to the extent that those performing the sampling repeat the process identically for each aliquot, for each outfall, and for each storm event. Automatic sampling standardizes the collection method and ensures repeatable, comparable data from sample to sample.

Withstanding regulatory scrutiny. Sample collection must produce results that accurately reflect the source. Sample variation can present problems. If the stormwater sample is "cleaner" than the actual storm discharge, the permit agency may assume that the facility is capable of maintaining that level of abatement and control. On the other hand, submitting samples that show excessively high contaminant levels could lead to requirements for unnecessary and expensive stormwater treatment systems. Thus, dischargers cannot afford to disregard the integrity of their monitoring efforts.

Sample collection time. While regulations require sampling only for the storm's first 3 hours, it is usually in the interest of the discharger to sample for the storm's duration. Because of the first-flush phenomenon (disproportionately high loadings in the storm's early stage), average loadings are lower and more accurately characterized with sampling over a longer period.

Avoiding repeated monitoring. The greatest cost associated with any sampling program is the cost of repeating work. Repeated trips to the monitoring site, lab analysis, and inaccurate information all increase the cost of stormwater monitoring and sampling. Skewed or inconsistent results usually mean that the monitoring has to be repeated. This may not be easy in characterizing storm events. ■



The Part 2 NPDES permit requires monitoring data for 3 storm events gathered from 5 to 10 outfalls.

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SEPTEMBER 1993

WATER ENVIRONMENT & TECHNOLOGY

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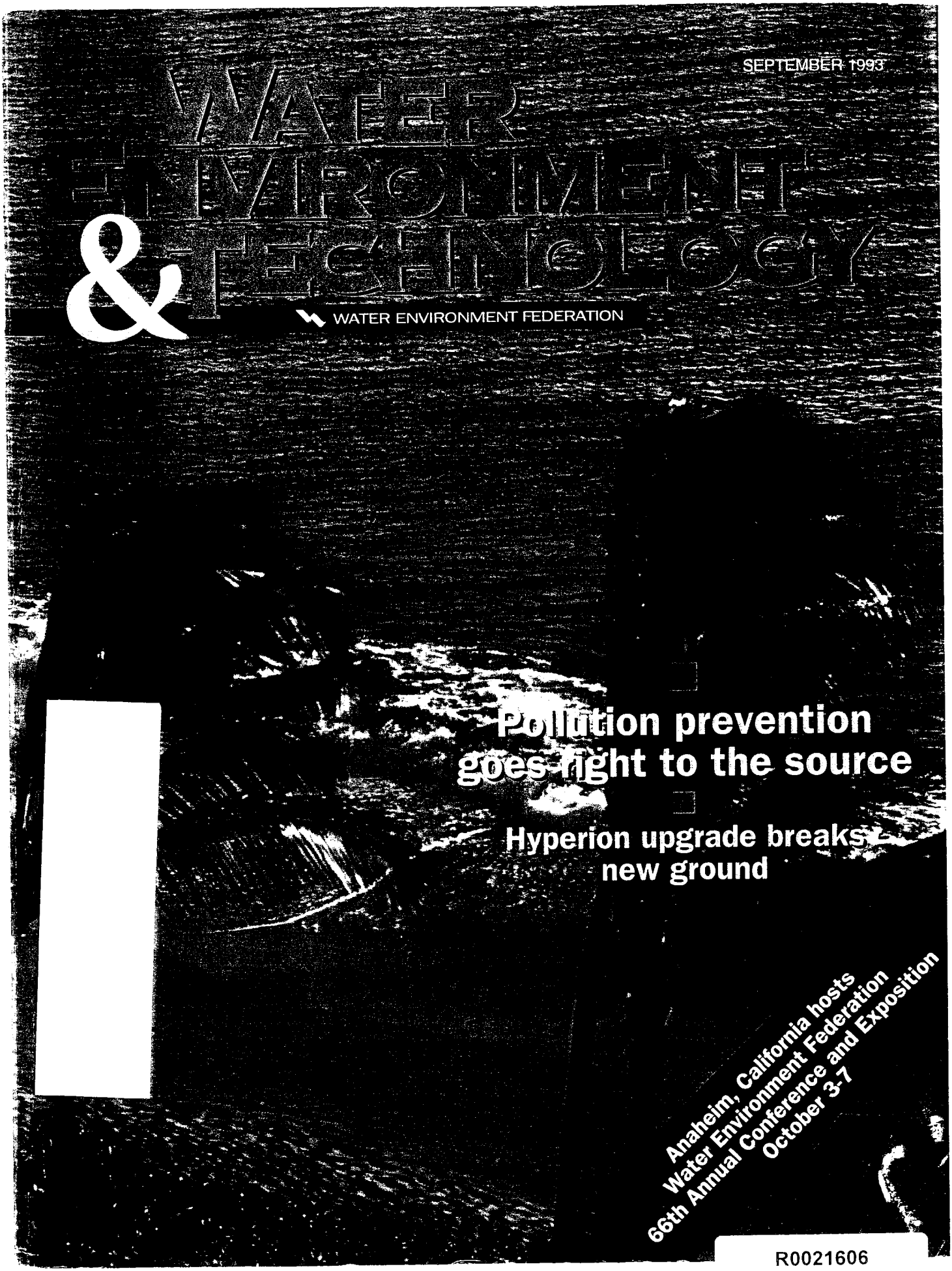

Pollution prevention goes right to the source

Hyperion upgrade breaks new ground

Anaheim, California hosts Water Environment Federation 66th Annual Conference and Exposition October 3-7

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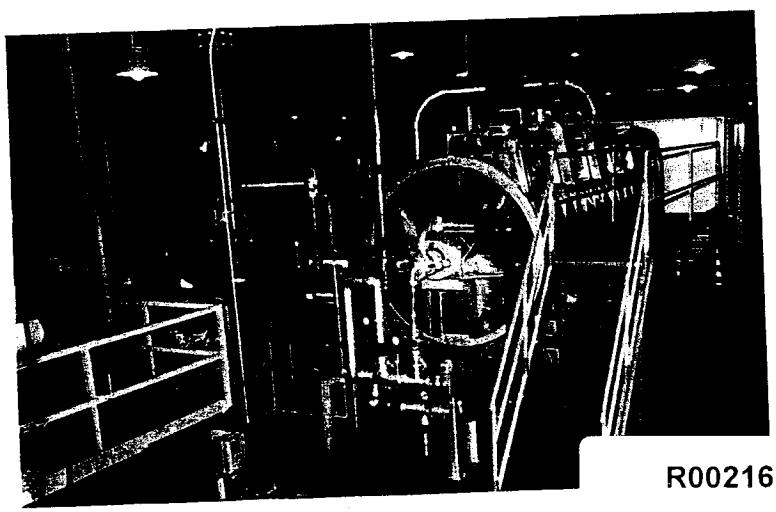

Installing a thermal dewatering system in an existing treatment plant can reduce operating costs.
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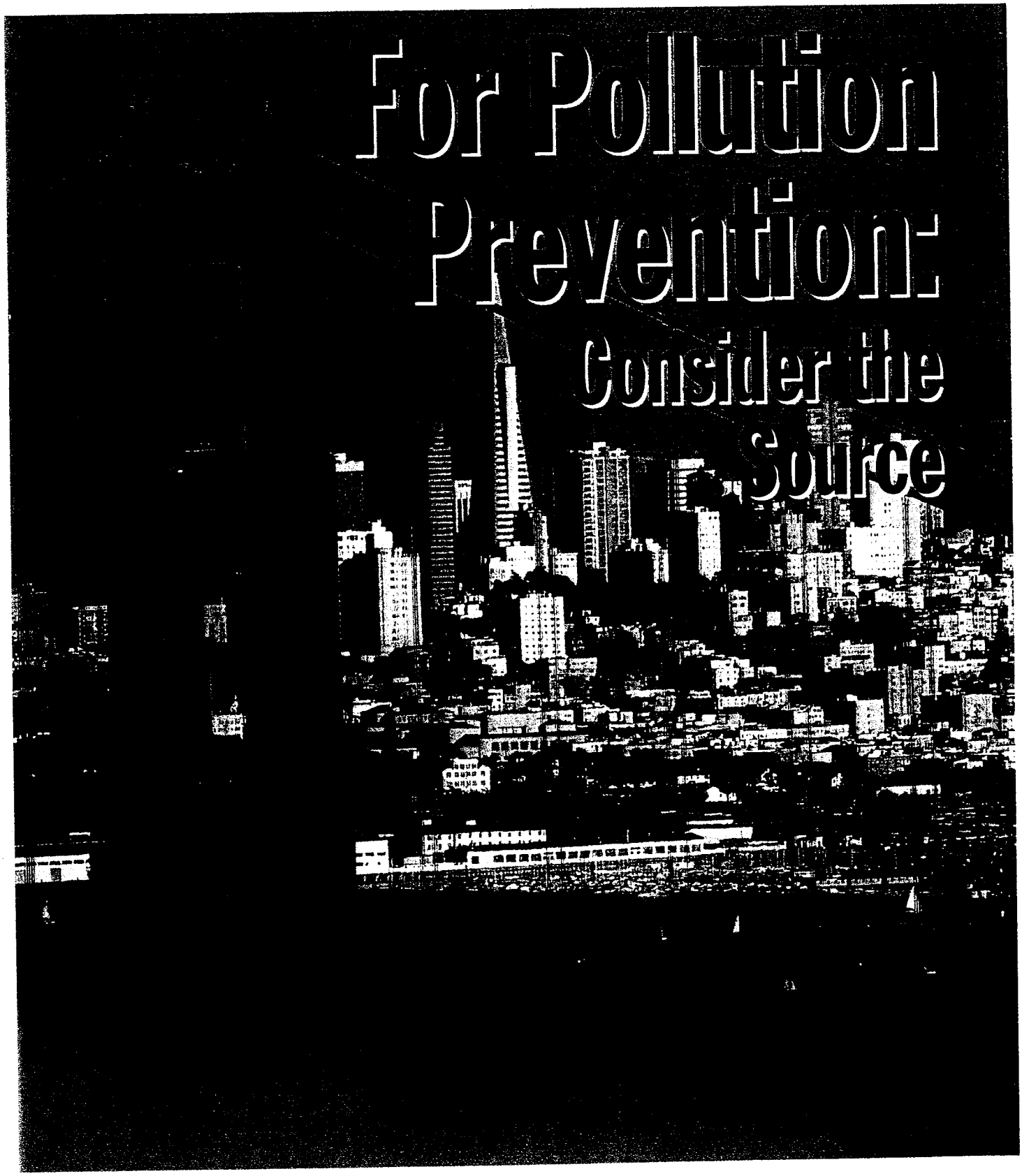
For Pollution Prevention: Consider the Source



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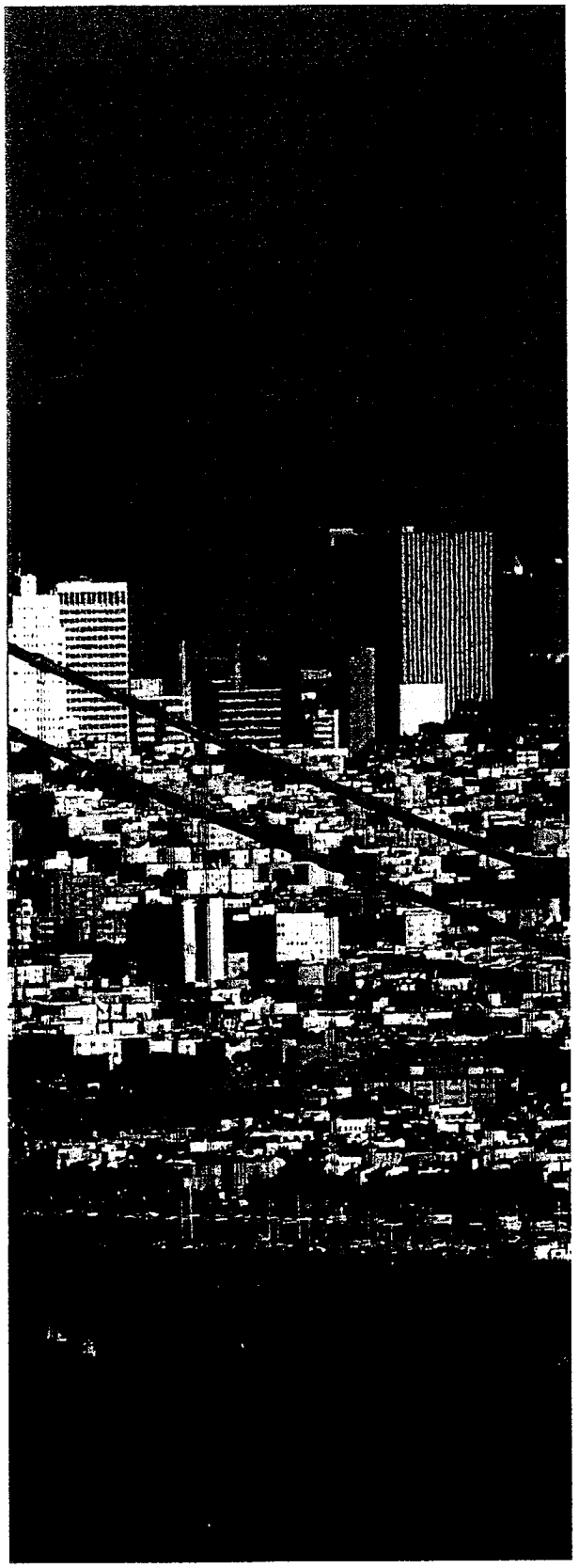

San Francisco's water pollution prevention program reduces cross-media transfer of waste

Frank Filice
Geoff Brosseau

San Francisco, Calif., has developed a water pollution prevention program that focuses on specific industrial, commercial, and residential activities to control pollutants at their source. The program provides an integrated, comprehensive approach to source control as well as providing departmental interaction, flexibility, and efficiency, which collectively reduce the risk of cross-media transfers of waste. This program adds another focus to the city's other hazardous material–waste minimization efforts.

San Francisco has a combined sewer system that serves a sewered area of approximately 12,500 ha (31,000 ac), with a resident population of 725,000. San Francisco has modest levels of precipitation [average annual rainfall is 0.53 m (21 in.)]. Before construction of combined sewer overflow (CSO) control facilities, overflows occurred on average between 46 and 81 times/yr, depending on the drainage basin. An average of 34×10^9 L/yr (9×10^9 gal/yr) of CSO was discharged before CSO control.

The Regional Water Quality Control Board [the state National Pollutant Discharge Elimination System (NPDES) permit issuing agency] required the city to reduce the number of untreated overflows to an annual average of 10/yr in the areas where shoreline usage is predominately maritime, 4 to 8 overflows/yr in areas where water-contact recreation occurs, and 1 overflow/yr near the shellfish beds along the southeast shoreline of the city.

The city is required to achieve compliance with their CSO control requirements by January 1, 1996. A series of transport and storage boxes with overflow wiers and two baffles, pump stations, and treatment plant improvements has been designed and constructed. The system provides secondary treatment to all dry-weather flows and a combination of primary and secondary treatment for wet-weather flows generated within the city.

Direct discharge from San Francisco's combined sewer facilities contains pollutants typical of urban runoff and domestic and industrial wastewater.

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A consumer product inventory showed that 35% of arsenic and 7% of chromium in the residential wastestream are associated with household products.

The NPDES permits also contain a requirement that effluent meet all existing and future water quality standards. In addition, the permits require monitoring of both effluent and receiving water for conventional and toxic pollutants. For CSO structures, no specific numerical limitations have been adopted; instead, the city must develop industrial source control programs and implement best management practices (BMPs) to minimize the introduction of toxic pollutants and monitor CSO discharges.

The city's first step was to quantify specific wet- and dry-weather pollutants of concern and their sources. Based on the results of these studies, the city's pretreatment requirements for industrial dischargers have expanded to include previously unregulated commercial and industrial dischargers. In addition, BMPs have been developed for residential activities.

DRY-WEATHER POLLUTANTS OF CONCERN

The pollutants of most immediate concern are those that may cause compliance problems with the city's wastewater discharge requirements, including:

- contaminants exceeding NPDES permit limits,
- contaminants contributing to general receiving water degradation (for example, contaminants exceeding receiving water quality objectives),
- contaminants in compliance but near permit limits,
- toxic contaminants that bioaccumulate or biomagnify, and
- contaminants contributing to sediment toxicity.

Copper and mercury concentrations currently exceed objectives of the San Francisco Bay Basin Plan, and cyanide concentrations occasionally exceed these objectives. An evaluation was performed on wastewater influent to the city's treatment plants to determine the metal loading associated with the potable water supply, as well as residential, commercial, and industrial areas (see Table, *Principal Dry-Weather Sources of Pollutants*).

Based on this influent data, processes and activities that have the potential to discharge pollutants of concern were investigated. In general, wastes from small, unregulated generators and residences are present in liquid form. These include inks, oils, and sol-

Principal Dry-Weather Sources of Pollutants

Metal	Water supply	Residential	Regulated industries	Other commercial services
Copper	Raw water supply Pipeline corrosion	Root killer products Algicides Foods	Manufacturers of electrical equipment	Auto body shops Cooling towers Commercial printers Linen suppliers
Mercury		Paints Photo wastes Auto fluids Foods (meats, vegetables, and grains)	General medical and surgical hospitals Lamp manufacturers	Paints and wallpaper Photo laboratories Auto services Metal manufacturers Laboratories Dentists
Lead	Raw water supply Pipeline corrosion	Paints Photo processors Auto fluids Home plumbing	Hospitals Radiator repair shops Linen suppliers Car washes	Sheet metal shops Auto services Equipment renters Painting services
Nickel	Raw water supply Pipeline corrosion	Paints	Metal platers Hospitals	Auto services Cleaners and laundries
Silver		Photo processors Personal care products Polishers	Photochemical suppliers Hospitals Motion picture industry	
Zinc	Corrosion prevention	Paints Auto fluids Water heaters	Hospitals Sausage manufacturers Laundries and linen suppliers	Auto services Printers
Cyanide			Silverware manufacturers and plating works	Motion picture industry Equipment renters Commercial laboratories

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vents. These small quantity generators can have a significant impact on the discharge of pollutants into the combined sewer system. Field studies of several large dental facilities showed significant contributions of mercury, silver, and to a lesser degree, copper and zinc to the metals loading identified in the influent to the treatment plants.

A consumer product inventory showed that 35% of arsenic and 7% of chromium in the residential wastestream are associated with household products. Household products also account for between 0.45% and 3.83% of total silver, nickel, cadmium, zinc, mercury, lead, and copper in the residential wastestream.

Mercury. The painting industry contributes significantly to mercury discharges. Phenyl mercuric has been identified in adhesives used by painting contractors. In addition, chloromethyl propyl mercuric acetate was found in adhesives, and in some spackling pastes and paint preservatives. Compounds found in paint-related products can reach the sewer system in many ways. For example, paint mixing and equipment washing result in wastewater that contains mercury. The tool rental industry is also a significant discharger of mercury because they lease paint-spraying devices. When such devices are returned without being cleaned, the rental facility must wash them.

Dental facilities and hospitals use mercury in amalgam-filling processes and in equipment such as chest suction mechanisms. Mercury is pumped through the equipment, and when the process is complete, the mercury is decontaminated for reclamation. The potential for accidental spills exists in these facilities.

Copper. The corrosion of copper pipes contributes significantly to total copper at the treatment plant. Copper is also found in cooling tower blowdown from heating, ventilation, and air conditioning systems used in a variety of high-rise buildings, manufacturing activities, and large retail facilities.

Copper is also found in commercial printing operations, radiator shops, root killers, and in algicides used to protect San Francisco's water supply.

Cyanide. Cyanide was evaluated by several different methods. Potassium ferricyanide is used in many different industries including film developing, which uses a bleaching solution that is partly composed of potassium ferricyanide. Unless a facility has an on-site reclamation program, the solution is discharged to the sewer system after use.

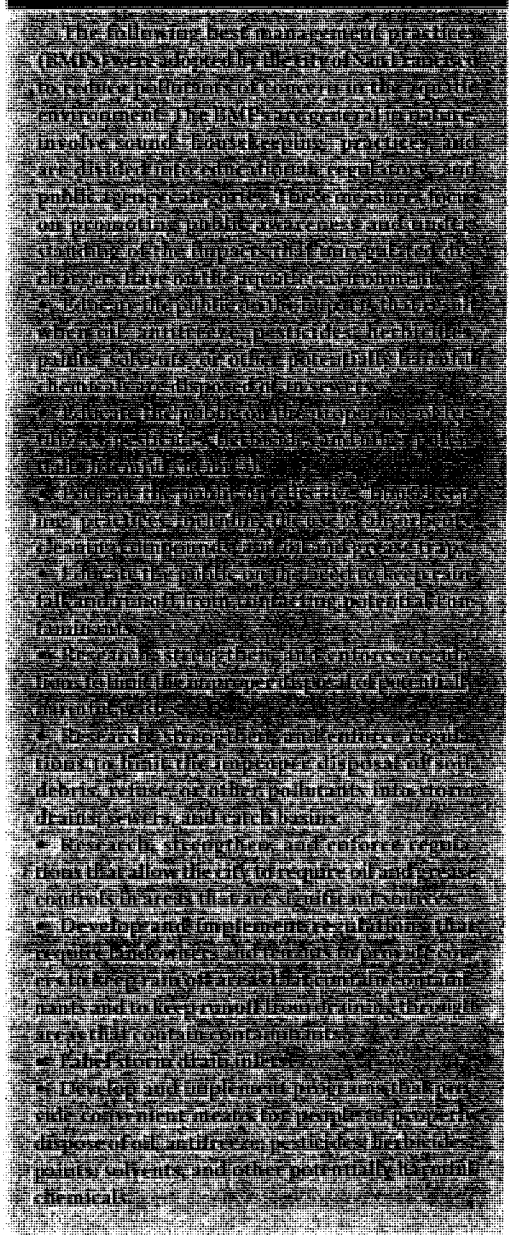
Cyanide is contained in certain dyes for ink, paints, and blueprint paper, and is also found in cleaning and laundry detergents, acrylics, and in the raw material used in manufacturing concrete.

Silver. Silver is found on film and photo paper. The silver enters the solution in the development process, and may then be discharged into the sewer. Silver is also used in dental offices, labs, jewelry making, and plating shops. The devices used to clean teeth leave trace amounts of silver in the mouth that are washed down the drain when a patient rinses. Silver may also enter the sewers during resilvering and replating.

WET-WEATHER POLLUTANTS OF CONCERN

As with dry-weather discharges, the major concern with wet-weather discharges is those pollutants that violate the city's discharge permits. Although the city's wet-weather discharges are currently in compliance, this situation may change as new water-quality objectives are

Pollution Prevention Pointers



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Principal Wet-Weather Sources of Pollutants

Constituents of concern	General wet-weather sources	Commercial and industrial wet-weather sources
PAHs[a]	Asphalt Gasoline Oil and grease Tires Wood preservatives	Auto service industry [illegible]
Copper	Algicides Brake linings Paints and wood preservatives Vehicle engine parts	Auto service and repair Electrical equipment manufacturing and metal finishing Paints, inks, and printing
Lead	Batteries Gasoline Paints Vehicle exhaust Vehicle engine parts	Auto service and repair Equipment rental
Cyanide	Animal wastes Cyanogenic insects Microorganisms Vehicle exhaust	Printing and publishing Metal finishing Silverware manufacturing
Cadmium	Gasoline Oil and grease Tires	Auto service industry Metal finishing
Mercury	Paints	Printing and painting contractors
Nickel	Gasoline Paints Oil and grease Brake linings	Local trucking Auto service industry
Zinc	Atmospheric fallout Galvanized iron and steel Oil and grease Tires Paint	Auto service and repair Commercial printing Equipment rental Local trucking
Chromium	Engine parts Brake linings	Auto service industry Metal finishing

[a] polynuclear aromatic hydrocarbons

incorporated into permits as discharge limits. The city conducted studies that determined its wet-weather pollutants of concern. Based on the last three years of overflow data, pollutants of concern were divided into three tiers:

Tier 1—significant concern. Three-year mean overflow pollutant concentrations that are 10 times greater than *potential* water quality concentration objectives without dilution (PAHs, copper, lead, and cyanide).

Tier 2—moderate concern. Three-year mean overflow pollutant concentrations greater than *potential* water quality concentration objectives without dilution (cadmium, mercury, nickel, silver, and zinc).

Tier 3—little concern. Three-year mean overflow pollutant concentrations less than *potential* water quality concentration objectives without dilution (chromium and arsenic).

The primary wet-weather sources of Tier 1 and Tier 2 pollutants of concern are automobiles and automotive-related industries. Pollutants are introduced when exhaust, leaking vehicle fluids, and corrosion of parts are deposited on streets. Pollutants of concern contact rainfall or runoff and are carried into the combined sewer system. Typically these pollutants include grease and oils, heavy metals, and PAHs (see Table, *Principal Wet-Weather Sources of Pollutants*). ■

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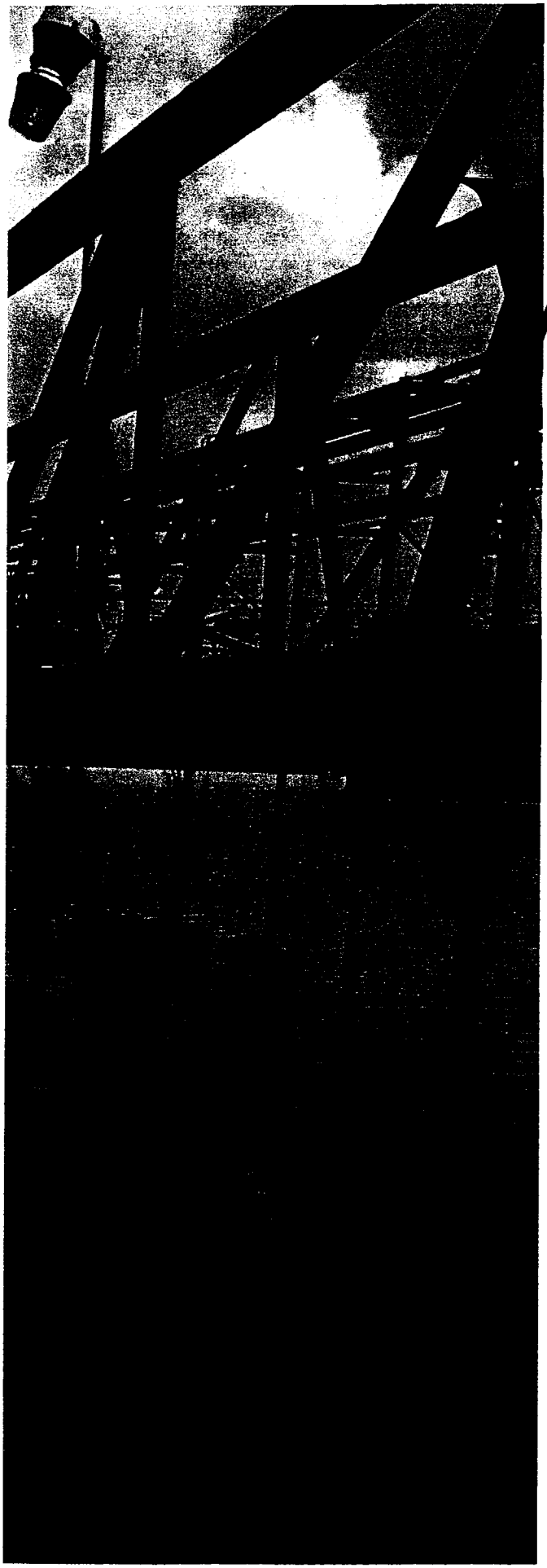

When it comes to pollution prevention and chemical manufacturers, the byword is *by-products.* Since 1987, the chemical industry has focused on using by-products before they become waste. Recently published findings from the EPA Toxics Release Inventory (TRI) indicate that the U.S. chemical industry has made significant progress in reducing toxic chemical waste. According to data compiled by the Chemical Manufacturers Association (CMA) from 1553 member facilities preparing TRI reports, the U.S. chemical industry recycled or treated 93% of the 5.9×10^9 kg (13×10^9 lb) of chemical by-products produced. The remaining 7% were reported to the TRI as permitted releases to the environment.

Survey respondents included manufacturers of industrial chemicals, plastics, manmade fibers, pharmaceuticals, soaps and toiletries, paints, and agricultural chemicals. These manufacturers have invested more than $7.3 billion since 1987 in pollution control equipment.

The TRI was established by the Emergency Planning and Community Right-to-Know Act and has been used since 1987 to track toxic chemicals at manufacturing facilities. The Pollution Prevention Act of 1990 has provided additional emphasis on source reduction. The inventory documents both yearly releases and off-site transfers of nearly 320 chemicals (see Box, *Targeted "33/50" Chemicals and Carcinogens Reduced*).

In the most recent inventory, toxic releases were down 6% from 1990; and, for the fifth straight year, releases were lower across the board. Air emissions were down 29%; water discharges, down 77%; land discharges, down 35%; and deep well discharges, down 35% from the base year, 1987. Chemical facilities also reduced off-site transfers of toxic chemicals for treatment and disposal by 37% (see Tables, *1991 Toxic Release Inventory* and *1991 Toxic Releases and Transfers by Media*).

Discharges from the wastewater treatment plant at the Dow Chemical Company's Plaquemine, La., facility account for the smallest portion of their Toxic Release Inventory report.

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1991 Toxic Release Inventory[a]

Year	1987	1988	1989	1990	1991	%Change
	Millions of pounds[b]					
Releases to land, air, or water	1307	1249	1085	899	848	–35
Transfers for off-site treatment or disposal	451	410	361	331	285	–37
Totals	1758	1659	1446	1230	1133	–36

[a] Chemical Manufacturers Association (1993) "Toxic Release Inventory Declines for Fifth Straight Year," *Pollution Prevention Fact Sheet*, p. 1.

[b] lb × 0.453 = kg.

Targeted "33/50" Chemicals and Carcinogens Reduced

The chemicals identified in EPA's Toxic Release Inventory vary widely in toxicity and in the amount released. However, release does not equal exposure. According to EPA, each release scenario has its own unique characteristics. Dilution may render exposure concentrations so small as to be of no concern. On the other hand, some environmental transformations lead to by-products of even greater concern than the original release. To set priorities, EPA and industry have used two measures: targeted pollutants and carcinogens.

EPA asked industry to cut releases of 17 large-volume chemicals by 33% voluntarily from 1988 to 1992 and by 50% in 1995. These are known as the "33/50" chemicals.

The Chemical Manufacturers Association (CMA) endorses the program because it is compatible with the goal of "responsible care," the industrywide initiative in health, safety, and the environment. Participating CMA member companies are 1 year ahead of EPA's goal, having achieved a 35% reduction in releases and transfers by the end of 1991.

Carcinogenicity, or the potential to cause cancer, is the other measure used by industry and EPA to set priorities for release reduction. Less than 5% of toxic releases and transfers from CMA core chemicals are considered carcinogens. From 1987 to 1991, releases of these chemicals have been reduced by 45% and off-site transfers are down 20%.

KEEPING OUTPUT UP AND RELEASES DOWN

The key for chemical manufacturers has been to find ways to cut chemical waste without sacrificing production. One prime example is the 889-ha (2200-ac) Monsanto Company plant in Pensacola, Fla., which produces nylon for carpets and industrial tires. The plant was one of the largest releasers of TRI chemicals in Florida, even while running at less than capacity during the base year, 1987. In 1988 and 1989, as production increased, so did releases. However, in 1990 and 1991, production continued at increased levels, but reportable releases were cut by 74%. The Pensacola plant achieved these reductions with several process changes.

A process change in 1989 reduced the amount of ammonia required to neutralize nitric acid, a by-product of nylon production. This, in turn, reduced the volume of ammonium nitrate disposed through deep-well injection by 8.2×10^6 kg (18×10^6 lb). As a safety measure, a new ammonia storage tank that reduced air emissions was also installed.

Additional process modifications and operational changes at the Monsanto plant from 1989 to 1991 cut releases of cyclohexane by 96%, and reduced releases of maleic anhydride, a chemical used in the fiberglass and food industries. In 1990, the plant also substituted other solvents for chlorofluorocarbons and methyl chloroform in degreasing and cleaning operations.

BUILDING A COMMITMENT TO POLLUTION PREVENTION

As a result of the Pollution Prevention Act of 1990, the TRI also now includes reporting of pollution prevention activities, such as recycling, treatment, and energy recovery at manufacturing sites. According to the TRI, chemical plants recycled 54% of the chemical by-products produced, and treated 31% in wastewater treatment units or disposed of in EPA-approved incinerators. Another 8% was recovered and consumed as energy in federally permitted boilers and furnaces. The remaining 7% was released directly to the environment (see Figure, *Percent of Chemical By-Products Recycled or Treated, 1991*). The reporting chemical facilities managed 5.5×10^8 kg (12.1×10^9 lb) of chemical by-products and released only 393.7×10^6 kg (868×10^6 lb) to the environment. Eighty-five percent of the chemicals recycled, treated, or recovered for energy use were managed at the plant site.

CMA POLLUTION PREVENTION CODE

To foster a commitment to pollution prevention among its members, the CMA adopted a Responsible Care Initiative that requires member companies to follow six codes of management practice including: *community awareness and emergency response, employee health and safety, pollution prevention, process safety, product stewardship, and distribution.* The Pollution Prevention Code is a measure designed to promote sound industry management of the residues that remain after reduction efforts. The code requires member facilities to evaluate reducing wastes and releases at their source first before evaluating recycling or treatment programs. CMA is also developing additional portions of the code that will address reviewing facility and contractor waste management practices, groundwater protection, and evaluating prior operating and disposal practices at active or inactive facilities.

POLLUTION PREVENTION SUCCESS STORIES

Pollution prevention often focuses on source reduction, and uses such techniques as in-process recycling,

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process modification, improved plant operations, raw materials substitution, and changes in end-products to reduce toxic waste.

In-process recycling can close the manufacturing loop. DuPont recently introduced a new technology to recycle the catalyst within the process of making an intermediate for elastomers and spandex fibers. The catalyst replaces fluorosulfonic acid, which is hazardous.

Raw material substitution is also a pollution prevention activity that is the focus of much technical effort in the chemical industry. Substitution is usually complex, but can be as simple as using a different glue. Monsanto and 3M replaced a glue that emitted 11,340 kg (25,000 lb) of volatile organic compounds (VOCs) with a water-based glue that emitted none.

1991 Toxic Releases and Transfers by Media[a]

	Millions of pounds[b]					
Year	1987	1988	1989	1990	1991	%Change
Releases to land, air, or water						
Air	545	503	469	426	389	[illegible]
Water	36	19	12	10	8	–77
Underground injection	677	674	548	430	420	–38
Land	48	52	37	34	31	–35
Transfers for off-site treatment or disposal						
Treatment	223	215	181	154	151	–32
Land disposal	63	52	43	36	32	–49
Publicly owned treatment works	165	143	137	140	115	–31

[a] Chemical Manufacturers Association (1993) "Chemical Industry Cuts Releases to Air, Land and Water," *Pollution Prevention Fact Sheet*, p. 2.

[b] lb × 0.453 = kg.

Source reduction techniques frequently work in tandem. In the late 1980s, PPG Industries, Inc., introduced a family of new resins for industrial paints with lower VOCs. However, the distillation step to lower the VOC content resulted in a solvent that was difficult to use or recycle. By 1991, PPG was generating 2.3 to 3.2 × 10^6 kg (5 to 7 × 10^6 lb) of the solvent annually. The company then modified plant equipment and processes to reformulate the resins and eliminate extraneous solvents. This made it easier to purify and recycle the solvent; the company was able to use 1.5 × 10^6 kg/yr (3.4 × 10^6 lb/yr) in a number of applications. Thus, to prevent pollution, PPG changed the end-products, modified processes and equipment, and recycled.

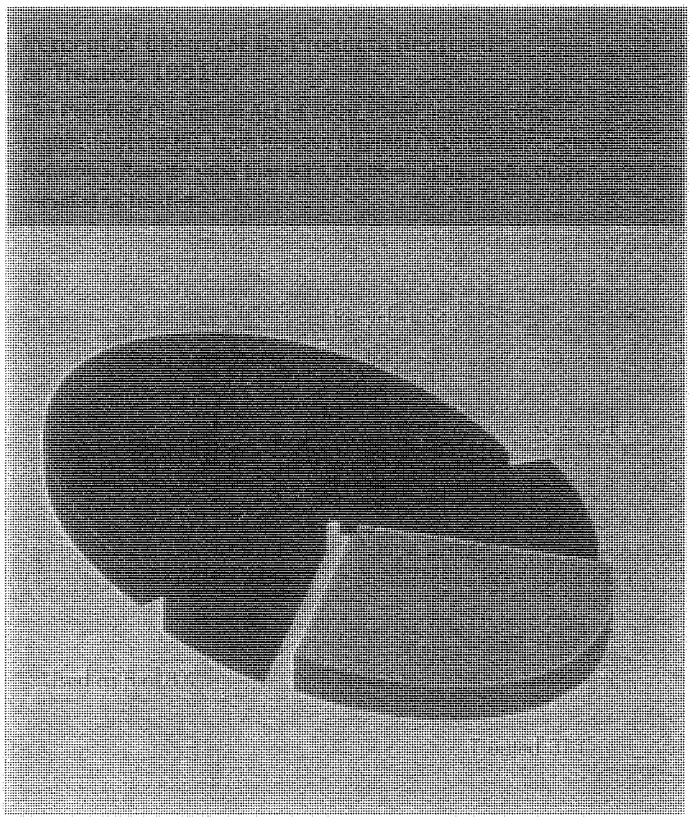
Recycling is a common pollution prevention activity. Air Products and Chemicals, Inc., recovered and recycled 272,160 kg (600,000 lb) of ammonia and ammonium nitrate from exhaust at its Escambia, Fla., plant. Air emissions were cut by 94%.

Two BASf Corp. coatings plants previously shipped solvents off-site for treatment. They began to distill the solvents for reuse in making paints, which reduced waste handling and saved $1 million/yr. In addition, residue remaining in the distillation units was used as fuel.

Velsicol Chemical Corp. also recovered the energy value of organic vapors from the manufacturing process at its Chattanooga, Tenn., plant. The organic vapors are burned to generate steam, thereby reducing air emissions by 95,256 kg/yr (210,000 lb/yr).

A biotank water treatment system at Georgia Gulf Corp's Plaquemine, La., plant uses naturally occurring organisms to convert contaminated water to carbon dioxide and water. Nearly 219 L/s (5 × 10^6 gal/d) of water are treated, including contaminated water from a new vapor recovery unit that reduced air emissions by 90%.

Other techniques to improve plant operations include preventive maintenance, materials handling procedures, employee training, and environmental reviews. A number of companies have improved internal and inter-industry inventory communications and control, particularly for small-volume chemicals such as those used in laboratories. ■



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R0021616

United States
Environmental Protection
Agency

Office of Research and
Development
Washington DC 20460

EPA/625/R-95/004
September 1995

DR

EPA

Seminar Publication

National Conference on Environmental Problem-Solving with Geographic Information Systems

Cincinnati, Ohio
September 21-23, 1994

ADMINISTRATIVE RECORD INDEX- DOCUMENT
STORM WATER TECHNOLOGY
FOLDER: 4, ITEM#: 4

R0021617

EPA/625/R-95/004
September 1995

Seminar Publication

National Conference on Environmental Problem-Solving with Geographic Information Systems

September 21-23, 1994
Cincinnati, Ohio

U.S. Environmental Protection Agency
Office of Research and Development
National Risk Management Research Laboratory
Center for Environmental Research Information
Cincinnati, Ohio

Printed on Recycled Paper

R0021618

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Please note that the Soil Conservation Service of the U.S. Department of Agriculture is now the Natural Resources Conservation Service; "Soil Conservation Service" is used throughout this document because this was the name in use at the time of the conference.

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Acknowledgments

The success of the conference and this document is due largely to the talents and efforts of many individuals. Gratitude goes to each person involved.

Authors

A special thanks goes to the many authors of the papers presented in this document. Their efforts in preparing papers made this document possible and led to the overall success of the conference.

Peer Reviewers

The following individuals peer reviewed this document:

- Jim Goodrich, U.S. Environmental Protection Agency, National Risk Management Research Laboratory, Cincinnati, Ohio.
- Nancy Phillips, Hollis, New Hampshire.
- John Shafer, Earth Sciences & Resources Institute, University of South Carolina, Columbia, South Carolina.

Editorial Review and Document Production

Heidi Schultz, Eastern Research Group, Inc., Lexington, Massachusetts, directed the editorial review and production of this document.

Technical Direction and Coordination

Daniel Murray and Susan Schock, U.S. Environmental Protection Agency, Office of Research and Development, National Risk Management Research Laboratory, Center for Environmental Research Information, Cincinnati, Ohio, coordinated the preparation of this document and provided technical direction throughout its development.

Special Thanks

Special thanks goes to the following individuals for their support:

- Tom Davenport, U.S. Environmental Protection Agency, Region 5, Chicago, Illinois.
- Mike Forrester, Urban and Regional Information Systems Association, Washington, DC.
- Bill French, American Society for Photogrammetry and Remote Sensing, Bethesda, Maryland.
- Carol Griffin, Idaho Falls, Idaho.
- Lyn Kirschner, Conservation Technology Information Center, West Lafayette, Indiana.
- Kevin Klug, Association of American Geographers, Washington, DC.

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- John Lyon, Ohio State University/American Society of Photogrammetry and Remote Sensing, Columbus, Ohio.
- Mark Monmonier, Department of Geography, Syracuse University, Syracuse, New York.
- Sam Noe, Joint Center for GIS & Spatial Analysis, University of Cincinnati, Cincinnati, Ohio.
- Nancy Phillips, Hollis, New Hampshire.
- Javier Ruis, U.S. Department of Agriculture, Soil Conservation Service, Fort Worth, Texas.
- Don Schregardus, Ohio Environmental Protection Agency, Columbus, Ohio.

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Introduction

The National Conference on Environmental Problem-Solving with Geographic Information Systems was held in Cincinnati, Ohio, September 21 to 23, 1994. The conference was a forum for over 450 environmental professionals to exchange information and approaches on how to use geographic information systems (GIS) to define, assess, and solve environmental problems.

Cross-media pollutant transport and watershed-based decision-making have made the process of solving environmental problems more complex. The application of GIS to environmental problem-solving has greatly increased our ability to manipulate and analyze relational and spatial data, providing environmental decision-makers with a powerful tool for analyzing multimedia environmental data over increasingly broader areas (e.g., watersheds, states, regions). While the approach to using GIS varies from application to application, a common, technically sound framework for applying GIS to environmental problems should be developed and implemented. This conference was an initial step in defining this framework by examining the following areas:

- Problem identification and definition.
- Data requirements (e.g., coverage, scale), availability, documentation, reliability, and acquisition.
- Approaches considered and selected for problem-solving.
- Unique challenges and pitfalls encountered.
- Interpretation of results, including level of confidence achieved based on data quality and approach taken.

Presenters were requested to address one or more of these areas in papers and posters that focused on applications of GIS to specific environmental problems.

This document presents peer-reviewed papers from the conference. The papers have been organized by general topic area as follows:

- GIS Concepts
- Ground-Water Applications
- Watershed Applications
- Wetlands Applications
- Water Quality Applications
- Environmental Management Applications
- Other GIS Applications

The purpose of this document is to share the information presented at the conference with individuals who were unable to attend. This document will be useful to individuals who are currently applying GIS to environmental situations or considering GIS for application in environmental problem-solving. These individuals include environmental regulatory personnel at the federal, state and local level; university professors, researchers, and students; private sector personnel, including industry representatives and environmental consultants; and other interested persons. The goal of sharing this information with a broader audience is to help users apply GIS to environmental problem-solving with a greater awareness of the power and limitations of this very useful tool.

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GIS Concepts

GIS Uncertainty and Policy: Where Do We Draw the 25-Inch Line?

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Abstract

The growing availability of improved hardware and software for geographic information systems (GIS) has outstripped most users' ability to identify and represent uncertainty in the available data. In practice, the proliferation and compounding of errors and uncertainty increase as information becomes more easily handled and combined from different sources.

Various stages of GIS database development and analysis generate different forms and amounts of error and uncertainty. In most cases, inherent uncertainty within source data is simply ignored and its nature eventually lost through subsequent processing. Both the location of features and their attributes can include error and uncertainty. By the time decision-makers receive mapped information, it is typically represented as correctly located and attributed.

The use of weather and climate information provided by the National Climatic Data Center (NCDC) is a common example of this scenario. Weather station locations provided by NCDC are reported to the nearest truncated degree-minute. A minute is one-sixtieth of a degree of arc. In the center of the continental United States, 1 minute of latitude averages approximately 6,000 feet and 1 minute of longitude averages approximately 4,800 feet. Thus, the station location is only known to lie within a box of approximately 1 square mile. Map representations of these data should reflect this uncertainty.

Under the Municipal Solid Waste Landfill (MSWLF) Criteria, the U.S. Environmental Protection Agency has dictated that the 25-inch precipitation contour line be used as a regulatory boundary for the level of protection required at municipal landfill sites. The way in which these lines are created and interpreted has important policy implications. Indeed, the cost and practicality of a given location must take this into account. If the 25-inch precipitation figure is critical, characterizing its uncertainty is also important.

In this work, uncertainty is considered a property of the data (1). A Monte Carlo procedure is used to represent the stochastic character of contour lines generated from point data with known locational uncertainty. The 30-year normal precipitation data for Kansas are used as an example. The results of this study are compared with the 25-inch contour used for regulatory purposes in Kansas. This study demonstrates that the method of interpolation greatly influences the resulting contours. In addition, locational uncertainty changes the results unpredictably using four different contouring methods. Finally, the differences have potentially significant policy implications. The nature and origin of these factors are discussed.

Problem Statement

The increasing power of geographic information systems (GIS) and the availability of digital data have enabled users and decision-makers to perform complex spatial analyses for a great variety of environmental applications (2). The rapid expansion of GIS has resulted in a parallel growing concern about the quality of data (3).

An understanding of error and uncertainty is critical for proper use of spatial information. For the purposes of this discussion, error is defined as a deviation between the GIS representation of a feature and its true value (4). For a location, this might arise from rounding or truncating digits. Attribute error can involve misclassification of a feature or some other form of incorrectly accounting for its nature. Error is a measurable value quantifying these differences.

Furthermore, uncertainty shall refer to a characteristic for which the exact location and/or quantity cannot be calculated (5) or an attribute whose value represents a distribution or some other ensemble (composite) measure. Locational uncertainty often arises when inappropriate measurement systems are used. An example of this is the use of a Public Land Survey designation (often

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referred to as a legal location) to specify a point location. This system is designed to represent a tract of land (an area). It is not accurate for locating points (1).

The uncertainty associated with an attribute is an important characteristic of that feature. It quantifies the precision of a stochastic quantity; that is, one that is not accurately represented by a single value. Annual precipitation is represented by a single number, typically the 30-year mean annual precipitation (30-year normal). This number varies each year, however, and that uncertainty can be quantified by the variance or other statistical measures. In this sense, uncertainty is a known or calculable value that can be used in spatial analyses.

Unreliable GIS data and products may lead to adverse environmental and legal consequences. The National Center for Geographic Information Systems and Analysis (NCGISA) chose data quality as the first initiative on its GIS research agenda (6). Many efforts have been made prior to this, and since, to understand and manage error and uncertainty in GIS applications.

GIS analyses are inherently subject to propagation of error and uncertainty (4, 7). No data set can represent every spatial reality of a geographically dispersed phenomenon. Monmonier (8) points out that as long as the three-dimensional earth's surface is transformed to a two-dimensional plane, error and uncertainty of various forms will be produced. Goodchild and Min-Hua (9) point out two issues that are important when dealing with error and uncertainty:

- Minimization of error in the creation of GIS products.
- Measurement and presentation of error and uncertainty in a useful fashion.

GIS technology introduces error and uncertainty through two major sources: (1) inherent error and (2) operational error. Inherent error is the error present in source data. It is generated when the data are collected. Operational error is generated during data entry and manipulation (7, 10-13). Examples include locational shifts due to projection or combining information from different source scales.

Most error and uncertainty contained in GIS data cannot be eliminated. Instead, they are actually created, accentuated, and propagated through GIS manipulation procedures (14-16). Most operational errors are difficult to estimate.

The selection by the U.S. Environmental Protection Agency (EPA) of a 25-inch per year local precipitation limit as one of the criteria to determine whether small municipal solid waste landfills (MSWLF) are subject to the provisions of Subtitle D provides an excellent example of how uncertainty and errors enter into a GIS analysis and its subsequent products. It demonstrates all of the major forms and purveyors of error and uncertainty:

- Spatial (locational) error
- Statistical (sampling) uncertainty
- Temporal (time domain) error
- Error proliferation (processing error)
- Analytical (choice of methodology) error
- Cartographic representation error

Many of these are avoidable; some are known and understood, yet they remain largely ignored by users of GIS technology. This work presents each of these factors, discusses their origins, and shows how GIS could have been used to better serve the policy and regulatory processes. The Kansas example demonstrates that ignoring the factors influencing error and uncertainty can result in incorrect conclusions and inappropriate policy decisions.

Data Requirements and Sources

To perform an analysis of precipitation, data are typically obtained from the National Climatic Data Center (NCDC), located in Asheville, North Carolina. This is the national repository for such data. These data are also available through state or regional climate centers. The Kansas Weather Library at Kansas State University provided data for this study. The 1990 "normal precipitation" data (17) and locations were obtained and generated into an ARC/INFO point coverage. Figure 1 displays the locations of the precipitation stations used in this study.

Normal precipitation is defined as the average annual precipitation for a three-decade (30 years) period at each station for which reliable data are available. To avoid "edge effects" (processing anomalies due to a lack of data along edges of an area), all stations in Kansas and some from neighboring states were used. A total of 380 stations compose this data set. In addition, precipitation contours from the "Availability of Ground Water in Kansas Map" (18) were digitized from a [paper] source map. The Geohydrology Section of the Kansas Geological Survey provided base map coverages of cartographic features.

All data represent the best available information from the source institutions noted above. Those organizations use the data in their analytical and cartographic research and production operations.

Methodology

To examine the influence of locational uncertainty on the representation of three-dimensional, natural phenomena, a Monte Carlo approach was adopted (1). Using this technique, random realizations of point locations are generated for each rain gauge, in each of 50 separate simulations. From this, 50 possible representations of the unknown locations of each gauge are used to create

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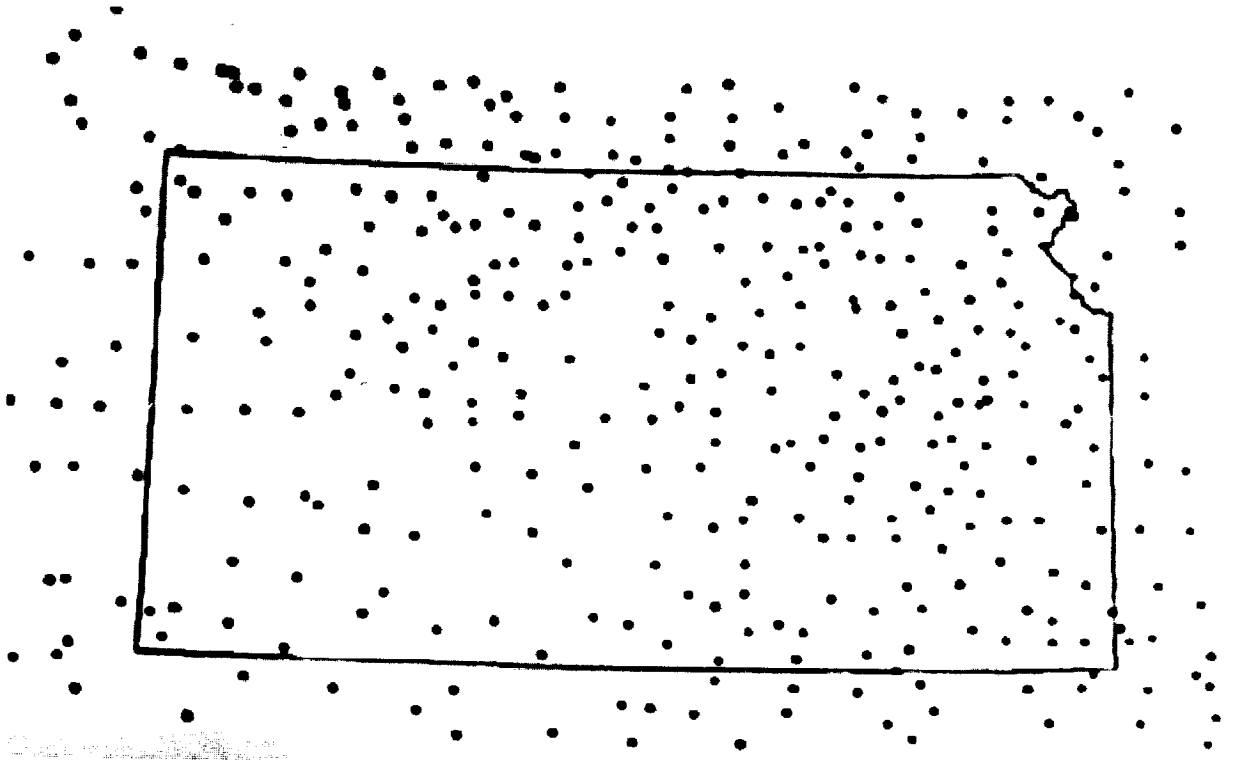

Figure 1. Location of rain gauges used in this study.

50 different sets of contours. All Monte Carlo calculations and data generation were performed using Statistical Analysis System (SAS) (19, 20).

These 50 simulations were sequentially processed using the four different contouring methods available within ARC/INFO. This provided a means to examine analytical error propagation. The first of the four methods is kriging (21, 22). This is referred to in the paper as the UK method, for its use of linear universal kriging (23). The other three are manipulations of the triangular-irregular network (TIN) contouring algorithm available in ARC/INFO. These differ by the number of interpolation points used along the edges of the elements in the TIN data structure (24). The first used the default 1, the second used 5, and the third used 10 (the largest value available). These are labeled D1, D5, and D10, respectively.

GIS operations used in this work include overlay analysis, areal calculation, and arc intersection. ARC/INFO was used for all GIS and cartographic production in this work.

Identifying the Sources of Uncertainty

Spatial Error

Data obtained from NCDC is provided with the knowledge that weather station locations are reported using truncated degrees and minutes of longitude and latitude. NCDC cannot provide any better locational accuracy at this time. Because each location is reported with error, this clearly has the potential to affect any contours or other three-dimensional features interpolated from the data. The magnitude and nature of this influence are unknown and unpredictable (1).

In addition to the poorly defined station locations, examination of the data revealed other anomalies. The locations in the publication reporting normal precipitation (17) were not identical to those identified by the Kansas State climatologist and NCDC. Some of the discrepancies were quite large. These anomalies were brought to the attention of all parties involved. No resolution was provided to this investigator's satisfaction, however.

The contours digitized from the "Availability of Ground Water in Kansas Map" are stated to originate from the 1960 normals (18). No documentation exists, however, concerning the way the lines were derived or the number of rain gauges used. Presumably, they were contoured by hand.

Statistical Uncertainty

This is a sampling consideration based on the size of the data, the nature of the process being sampled, and its variability. Unfortunately, precipitation is a particularly "patchy" phenomenon. That is, rain falls in a discontinuous fashion, and adjacent gauges can depict very different patterns. This is confounded by the fact that most contouring algorithms and other approaches

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represent three-dimensional surfaces assume a relatively smooth (locally) and continuous process.

Areal processes are almost always sampled as point information. Most contouring algorithms require a regular grid from which to interpolate surface features. In practice, rain gauges, as well as other environmental sampling programs, are irregularly distributed. Placement often depends on factors other than grid sampling (e.g., convenience, access to communications, finances, Congressional districts). This creates a "nonexperimental sampling" design (25). Nonexperimental sampling can contribute to uncertainty (26, 27).

Temporal Error

The normals are recalculated each decade and can change drastically in local areas. These changes arise for various reasons. First, some stations enter and drop from the database. Stations are deleted due to changes in location or extended periods of data collection problems. On occasion, new stations are added. Thus, the size and areal coverage of the data set changes with time.

In addition, weather patterns change with time. Extended periods of drought or excess rain or snow alter measured precipitation. In turn, three-dimensional representations change unevenly.

Error Proliferation

Once an error enters into the database and is included in GIS operations, spatial analysis, or spatial interpolation, its effect passes into the next stage of processing. In the 1990 normal precipitation data for Kansas, two stations are reported in Garden City. Despite the fact that the two are only a few miles apart, their annual total precipitation differs by 2 inches! In consultation with the state climatologist (Mary Knapp, Kansas State University), one was eliminated from the analysis. This process was repeated for an additional six stations where reported values appeared to be anomalous compared with nearby stations or the previous normal precipitation (1951 to 1980).

Errors can also proliferate through the normal handling of data. With geographic data, this often occurs while converting data from raster to vector and vector to raster forms (28). Some GIS operations are best accomplished in one form or another. As a result, transformations are often "hidden" from the user. Commonly, features "move" slightly after each step in an analysis.

Analytical Error

Different techniques have been developed for performing spatial interpolation, and an abundance of software is available for this purpose. All these methods have strengths and weaknesses. Each is based on a specific set of assumptions about the form and nature of the data. Some are more robust (less sensitive to data anomalies) than others. Most importantly, some provide additional information useful in data analysis.

Unfortunately, users often "take the defaults" when using sophisticated techniques and ignore the assumptions behind the method. Parameters can be varied and their effect evaluated, as in a sensitivity analysis (29). Often, the best approach is to try several methods and evaluate their joint performance (30, 31).

Another difficulty is the need to assign values to areas. By definition, polygons in a GIS are considered to be homogeneous. In reality, they bound areas that are a gradation from one characteristic to another. On the other hand, contours are commonly used to depict surface gradients but are useless (within a GIS) for analytical or modeling purposes. Ultimately, data sampling is accomplished as a point process (except, perhaps, in remote sensing), while many forms of data analysis and processing require areal information.

Cartographic Representation Error

Communicating the uncertainty of map features is not a trivial endeavor. Maps can be produced in two basic forms: as a raster (e.g., orthophotoquads or satellite images) or a composition of vectors (e.g., contour maps). The printing process, however, often reduces all of this to a raster representation at a very fine pixel size. Each method poses its own problems in depicting uncertainty.

Rasters can be used effectively in conjunction with color information theory to produce a continuum of shading within a thematic map layer (32). The choice of colors, however, can influence the interpretation of the data, and no universal scheme exists for depicting thematic variability. For example, blue shades often represent water or cold, while yellow and/or red often represent temperature or heat.

Vectors present a different suite of problems. Contouring is the primary technique for using vectors to depict areal variation. By definition, however, contour lines represent an exact isoline or single value along its length. Uncertainty cannot be represented in a line. Rather, a composite of lines can be displayed that represents a set of possible interpretations of the data. This is not a practical solution for mapping, however, as it can create a jumble of intersecting lines that makes interpretation difficult and is not an aesthetic means of presentation.

Challenges Encountered in This Study

This work attempts to discover and account for sources of error and uncertainty in GIS analysis. Given this information, the challenges are to find the best way to incorporate it into the analysis and to represent it in a useful manner. Another challenge is finding ways to use

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GIS uncertainty to support policy and management decisions. Addressing these manifold problems starts with identifying the sources of error and uncertainty, the way they enter the analysis, and the manner in which they are propagated through the use of GIS.

This study includes a number of known sources of uncertainty. In practice, this is not always the case. Users of GIS data and technology should always assume that the sources of uncertainty discussed in this paper are present and attempt to determine their nature. Uncertainty should be considered a property of the data and appropriately represented (1). This is the approach taken in this work.

After examining these factors, a Monte Carlo simulation was deemed the most appropriate approach to capture the nature of the locational uncertainty. Four different methods of contouring were used to examine analytical uncertainty (uncertainty due to the choice of a contouring algorithm). In addition, the contribution of statistical (sampling) uncertainty could have been addressed through incorporating information about the standard error of the point precipitation measurements (normals) used as the base data. Time limitations precluded examining this dimension of the question. Comparing the contours resulting from the 1960 and 1990 precipitation normals demonstrates the effect of temporal variation.

The greatest challenge is communicating the uncertainty in a manner useful to decision-makers. This paper presents a series of maps, figures, and tables aimed at addressing this problem. Some of the maps (see Figures 2 through 5) show the uncertainty resulting from each of the contouring methods. Figure 6 depicts the union (overlay) of the four approaches and displays their correspondence. The pie chart in Figure 7 is a nonspatial representation of this correspondence and the relative area represented within the different combinations of overlapping regions of uncertainty. Tables 1 through 3 further compare these quantities. Figure 8 is the map that the Kansas Department of Health and Environment (KDHE) chose to define the regulatory boundary (the 25-inch contour). The contours resulting from this study can be seen in Figure 9. Finally, the map in Figure 10 is a cartographic comparison of the differences between the contours used by KDHE (based on the 1960 normals) and those generated by the currently available data (the 1990 normals).

There is no single best approach for meeting these challenges, and there may never be one. The real challenge to address is how to educate technical GIS professionals and the users of their work to look for uncertainty and consider its influence on their decision-making process.

Results

The zones of uncertainty defined by the results from the four contouring methods used in this study are displayed in Figures 2 through 5. For each method, these zones represent the areal extent of the overlain contour lines produced in the 50 simulations. Each region is bounded by the furthest west or east contour generated along any length of the region. Table 1 shows the relative area falling within each of these zones as they traverse the state of Kansas. Clear differences exist between the total areas of uncertainty. It is their placement and relative location, however, that have policy and management implications. GIS is required to examine these questions.

Table 1. Comparison of Absolute and Relative Area of Uncertainty Arising From Four Methods of Determining the 25-Inch Precipitation Contour

Method[a]	Total Area (square miles)	Difference Between This and UK Method (square miles)	Percentage of UK Method	Percentage of Combined Area[b] of Uncertainty
UK	289.33	—	—	19.65
D1	494.38	205.05	170.87	33.58
D5	602.13	312.80	208.11	40.90
D10	631.11	341.78	218.13	42.87

[a]UK = universal kriging with linear drift, D1 = TIN interpolation with 1 subdivision, D5 = TIN interpolation with 5 subdivisions, D10 = TIN interpolation with 10 subdivisions (23, 24).
[b]A union (overlay) of all four sets of regions of uncertainty creates a combined area of 1,472.07 square miles. This includes the zones of uncertainty for each method of contouring and areas not included within any of the four regions of uncertainty (gaps between them).

Figures 2 through 5 clearly show differences in both the extent of uncertainty in the 25-inch contour line and its positional interpretation. Each method has a slightly different bend or twist. Islands (isolated regions where the 25-inch line appears as a closed loop) are manifested differently depending on the interpolation scheme. It is interesting to note the relative correspondence between the general shape of the D1 and UK methods. In the south-central border region, D1 and UK represent the local uncertainty as a bulge, while D5 and D10 depict it as an island of lower precipitation.

Areal correspondence and difference are depicted in Figure 6 and Table 2. Figure 7 is a pie chart visualizing the information in Table 2. These results are somewhat surprising in that areas where none of the four methods located the 25-inch line ("None Present") represent the second largest composite area. Because of the large number of "sliver polygons," the graphic representation of the overlay is somewhat difficult to interpret. Table 2 clarifies these interrelationships by breaking down the various categories. The average area per polygon value

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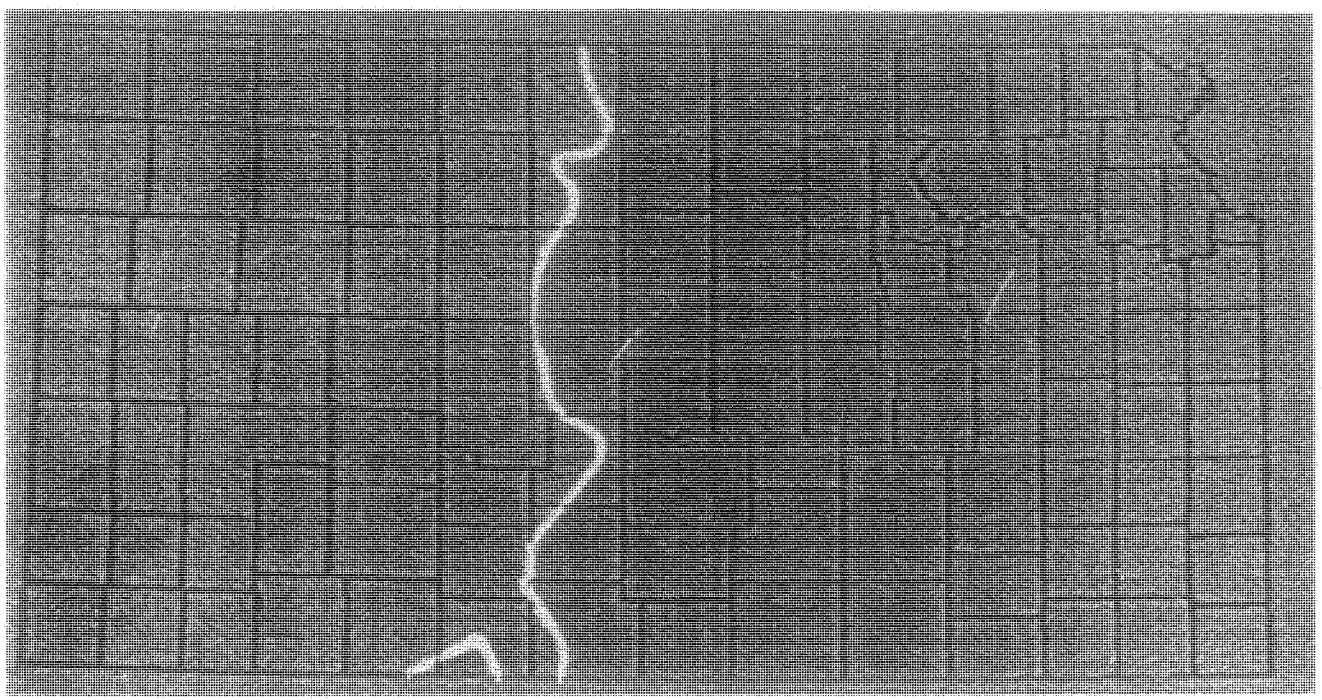

Figure 2. Regions of uncertainty produced by the UK method of contouring.

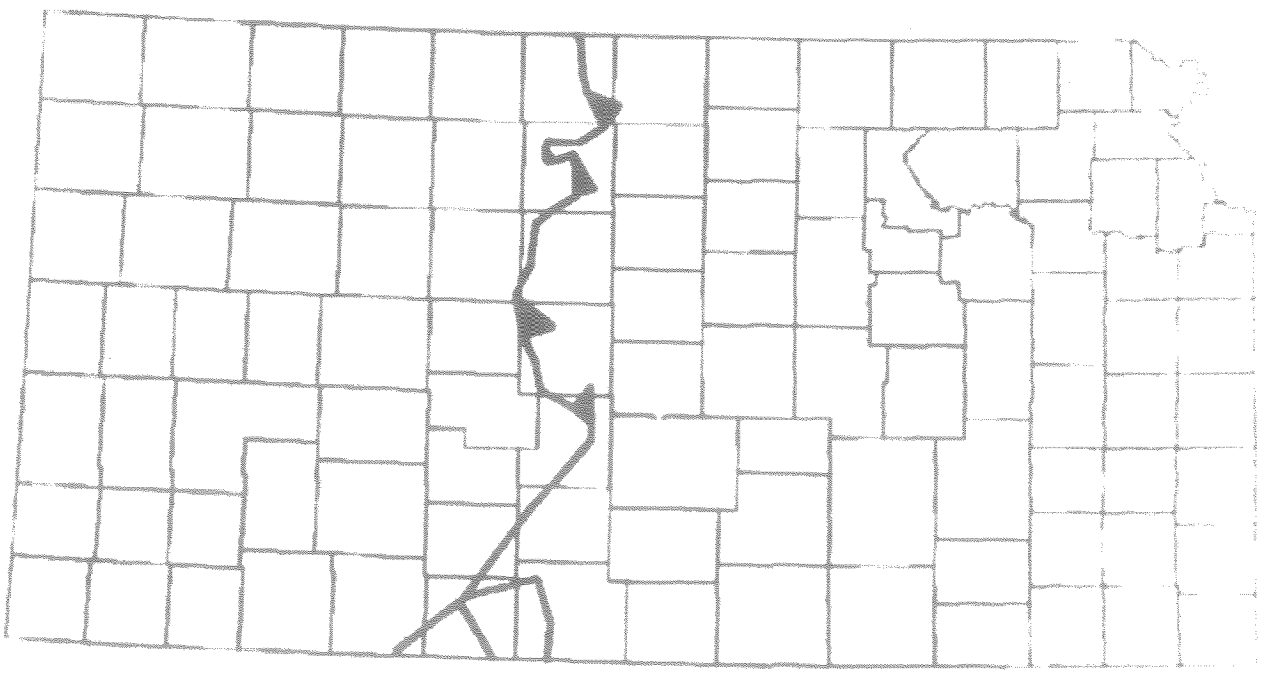

Figure 3. Regions of uncertainty produced by the D1 method of contouring.

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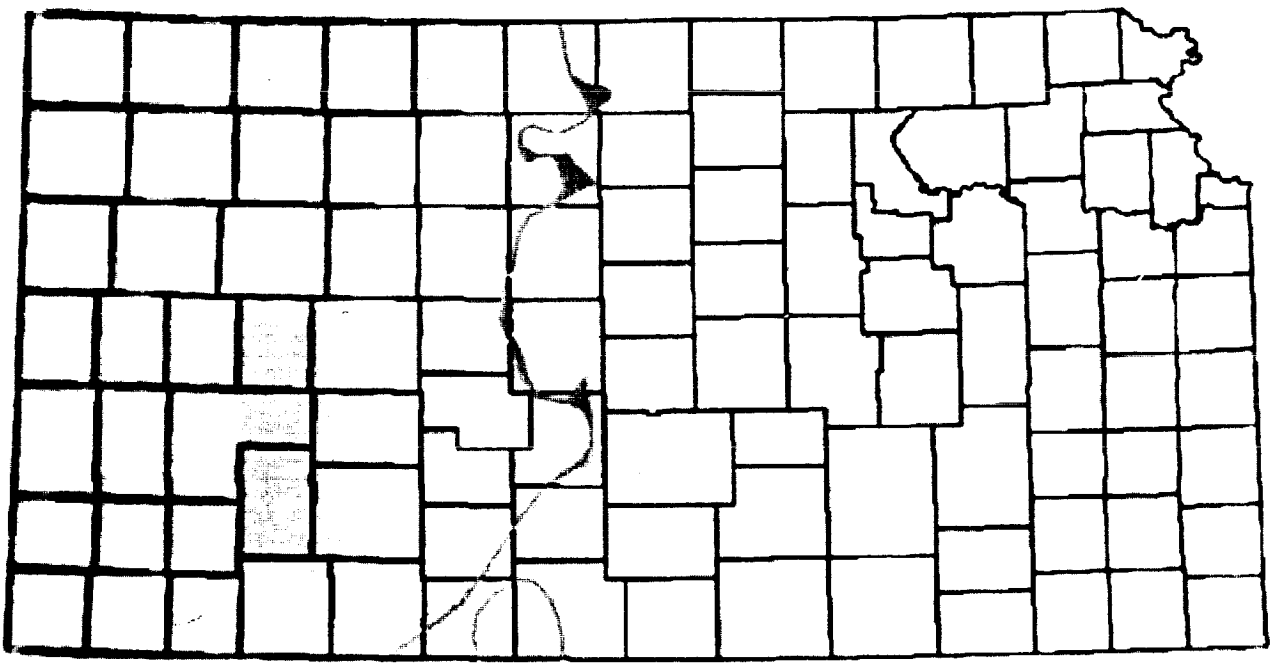

Figure 4. Regions of uncertainty produced by the D5 method of contouring.

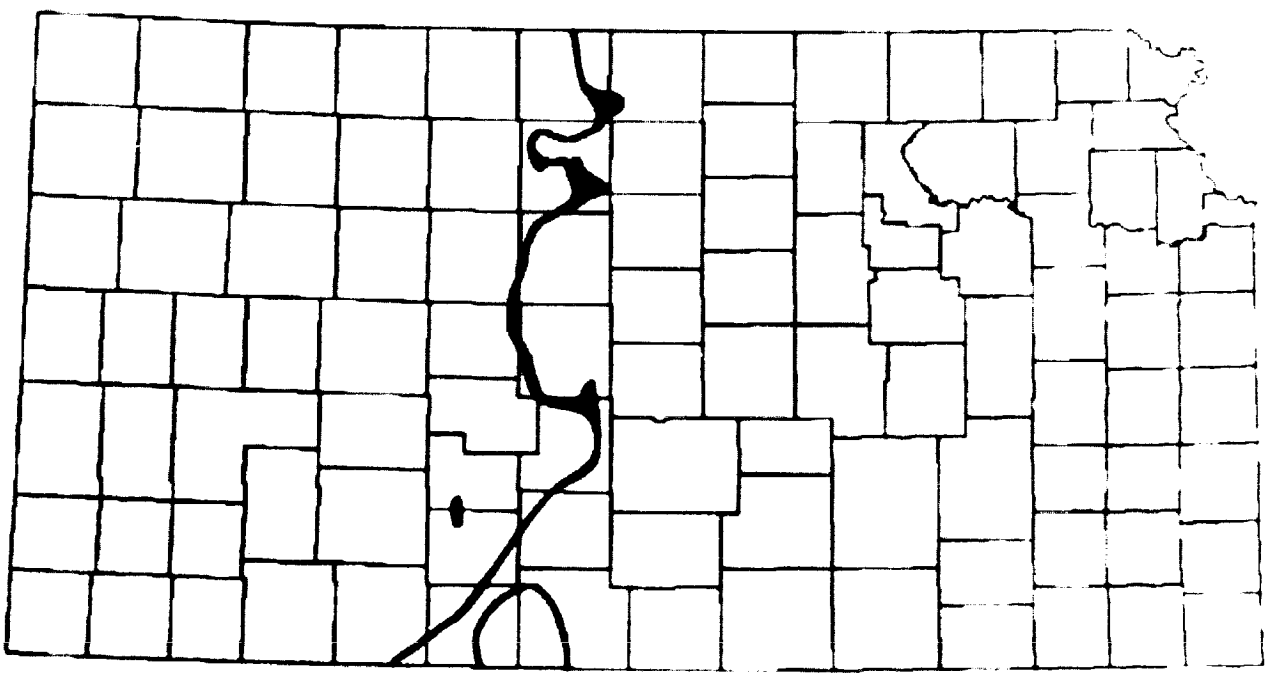

Figure 5. Regions of uncertainty produced by the D10 method of contouring.

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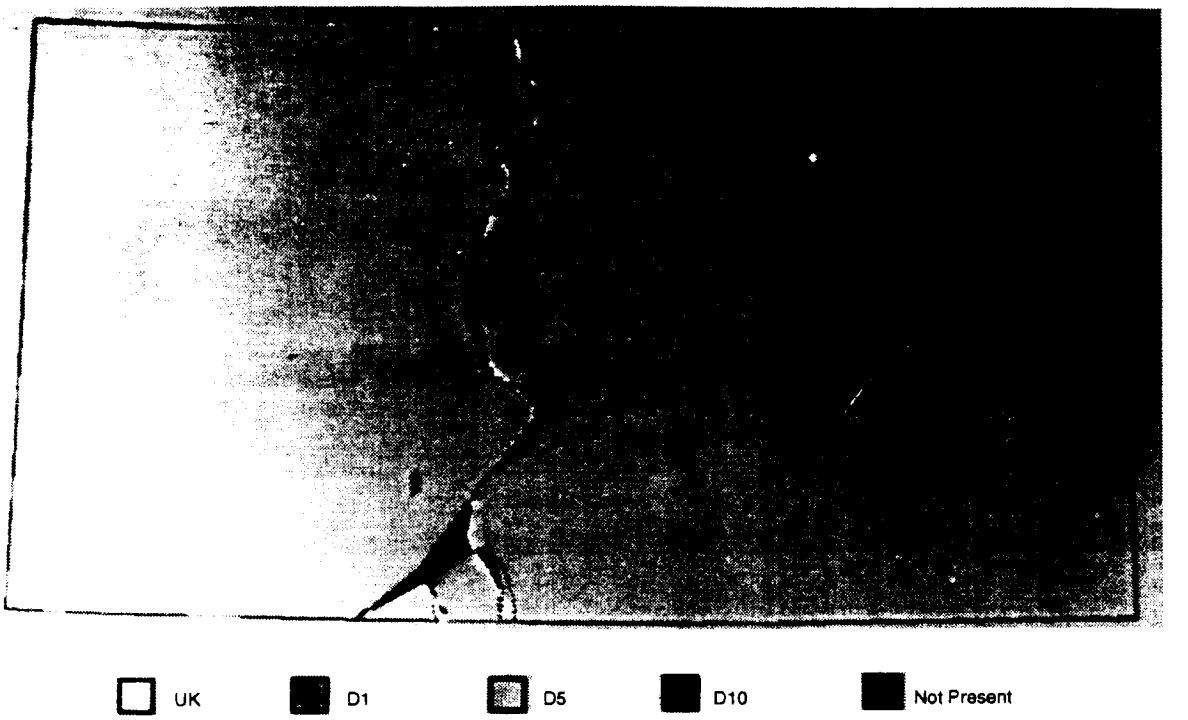

Figure 6. Union of the regions of uncertainty from all four methods of contouring. The numerous "sliver" polygons make this a difficult presentation to interpret at this scale. The black areas appear prominently, however. These represent areas where no method placed contour lines.

adds to the interpretation of the relative areas by incorporating the number of polygons in each category. An inspection of this column makes those categories with a multitude of very small polygons stand out. It also displays a number of large jumps in magnitude. As this number increases, the significance of the correspondence increases.

Table 2 highlights the correspondence between the D5 and D10 approaches and the D1 and UK methods of interpolation. This relationship is interesting because the algorithms used by the UK and D1 methods both are forms of linear interpolation. The D5 and D10 algorithms are designed to provide more "smoothing" and appear to create increasingly more "bull's eyes." Only D5 and D10 generate these features. Of the 50 simulations, a particular bull's eye appears west of the 25-inch contour (see Figures 3 and 4) four times using D5 and 39 times using D10. The size and location of these anomalies also vary with the input data. Polygons containing contours from all four methods rank ninth in total area and seventh in average area (out of 17 categories). This supports the conclusion that the four chosen methods have a relatively low spatial correspondence.

Table 3 breaks down the area of uncertainty by county. Although the zones of uncertainty appear to be relatively small when displayed on a statewide basis, they have important impacts in local areas. In particular, combining this information with soils, topography, ground water, and other information can clearly indicate whether a specific location is suitable for a landfill. Often, information developed at one scale is used in another. In this case, statewide information is being used for a site-specific application.

Figure 8 is a copy of the map the KDHE used to delineate the 25-inch precipitation contour. The results from

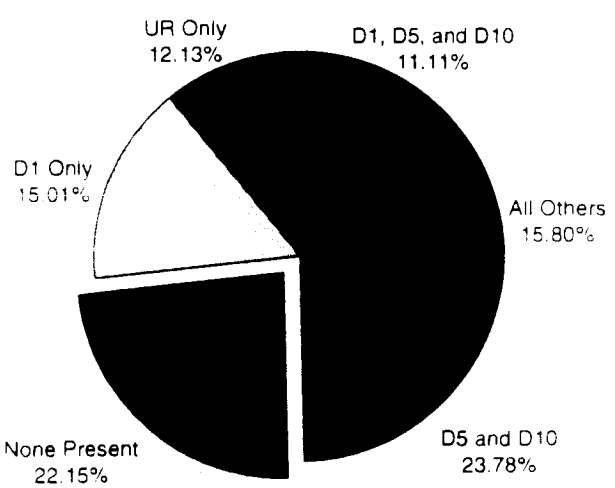

Figure 7. Breakdown of the total area in each category resulting from the union (overlay) of the regions of uncertainty from all four contouring methods. The "None Present" category represents a surprisingly large proportion among the 17 possible combinations.

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Table 2. Comparison of Absolute and Relative Area of Uncertainty Arising From Four Methods of Determining the 25-Inch Precipitation Contour

Methods of Contouring Found Within Area	N	Total Area (square miles)	Percentage of Total Combined Area	Average Area per Polygon[a] (square miles)
D5 and D10	28	350.13	23.78	12.50
None present	28	326.06	22.15	11.64
D1 only	37	221.02	15.01	5.97
UK only	27	178.60	12.13	6.61
D1, D5, and D10	24	163.61	11.11	6.82
D10 only	69	70.07	4.76	1.02
UK and D1	19	65.82	4.47	3.46
D5 only	73	34.68	2.36	0.48
Common to all	10	23.45	1.59	2.35
UK, D5, and D10	14	15.47	1.05	1.11
D1 and D5	38	10.68	0.73	0.28
D1 and D10	38	6.50	0.44	0.17
UK and D5	19	2.21	0.15	0.12
UK, D1, and D5	12	1.89	0.13	0.15
UK, D1, and D10	12	1.41	0.10	0.12
UK and D10	13	0.47	0.03	0.04
Combined total	461	1472.07	100.00	

[a]Average Area per Polygon = (Total Area) / N. This a useful measure to compare the relative size of each polygon in each classification.

Table 3. Area of Uncertainty for Each County Arising From Four Methods of Determining the 25-Inch Precipitation Contour

County	UK Area (square miles)	D1 Area (square miles)	D5 Area (square miles)	D10 Area (square miles)
Barber	36.38	24.29	28.05	28.31
Barton	28.47	96.84	56.16	59.56
Clark	12.64	11.49	12.97	13.02
Comanche	44.30	52.94	51.43	52.23
Edwards	—[a]	—	—	9.44
Ellis	—	6.14	9.27	8.90
Jewell	—	6.65	23.05	24.26
Kiowa	15.15	14.32	17.81	31.12
Osborne	43.87	99.82	168.2	172.6
Pawnee	—	—	0.11	0.30
Pratt	18.84	11.30	11.56	11.43
Rush	0.04	8.51	31.32	29.51
Russell	27.17	22.71	28.30	28.00
Smith	27.59	68.47	67.38	66.34
Stafford	34.88	70.90	96.49	96.05
Total	289.3	494.4	602.1	631.1

[a]A dash indicates that no contours appeared in that county for the method specified.

the UK method were selected as the best available representation of normal precipitation across Kansas (see Figure 9). The figure displays the unclipped contour lines generated from the data. This is done to point out the importance of "edge effect." Note the incoherent behavior of the contour lines at their termini. If a smaller window of data points were used, interpolation problems would have lain across the region of interest. When present, these features require more handling and time for analysis. They often introduce additional error and uncertainty.

The policy implications of this example are demonstrated in Figure 10. Here, the map shows the combination of the "official" KDHE map and the data interpreted in this study. The pattern of noncorrespondence is noteworthy. The lightest areas are regions that currently experience higher annual precipitation than forecast by the 1960 normals (from the KDHE map). Black areas are expected to have lower precipitation under current climatic conditions. Therefore, large areas of Kansas that should be under regulation according to the MSWLF regulations are not.

In summary, the figures and tables clearly show that locational uncertainty of data measured as points is propagated into contour lines. The nature and magnitude of that uncertainty varies with location and method of interpolation and shows no regular (predictable) pattern. Perhaps most importantly, uncertainty that appears small at one scale can be relatively more significant at another. In addition, seemingly small geographic feature and uncertainty can be an important factor in decision-making.

Discussion and Conclusions

GIS is an established and accepted technology, especially in applications related to natural resource and environmental management. Despite the widespread proliferation of GIS into these areas, the available data are not always appropriate for the intended application. Furthermore, adequate documentation is not always available to determine whether the data are adequate for a given use. The development of metadata standards will play an important role in addressing this problem. Errors and uncertainty will always be present in GIS data. Recognizing their presence, incorporating them into the analysis, and representing them in GIS products will remain a constant challenge.

This study demonstrates the influence that various sources of GIS uncertainty can leverage on the results of an analysis. The example of the 25-inch precipitation

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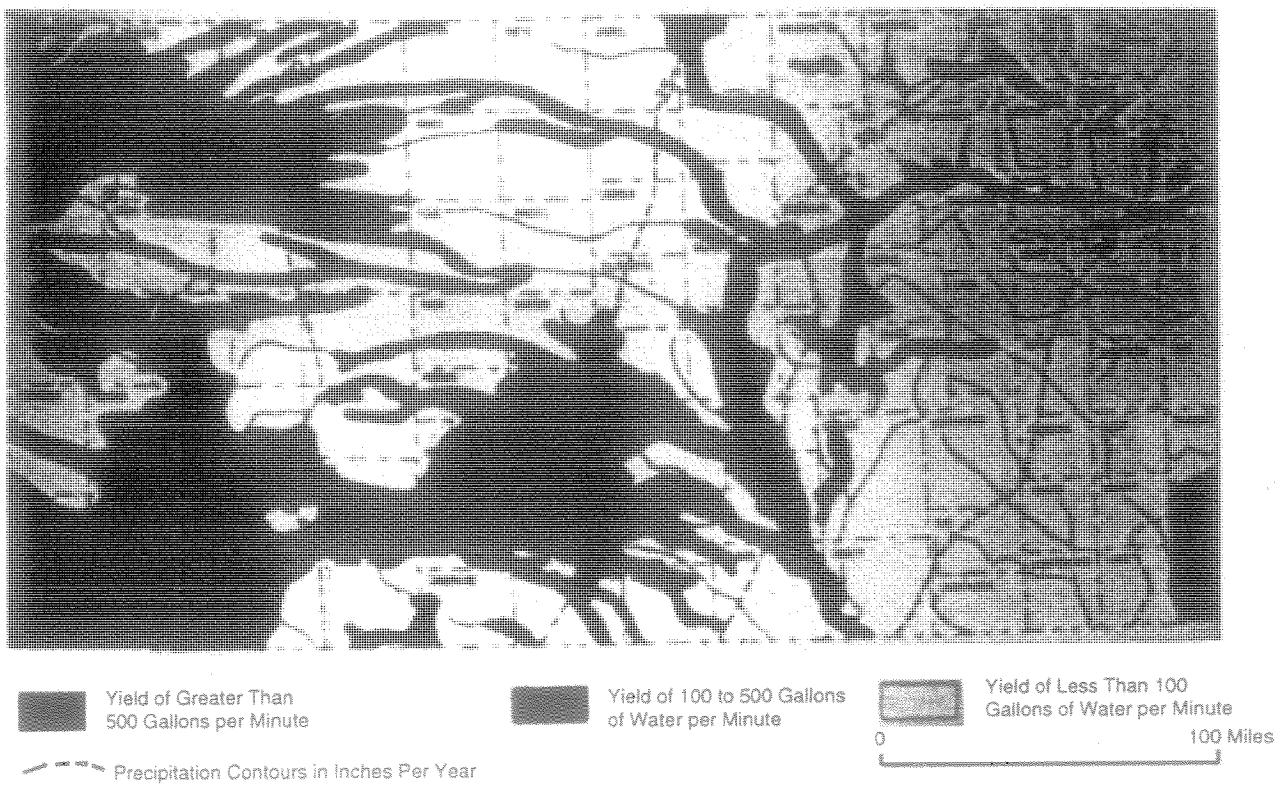

Figure 8. The map that the KDHE selected as the definitive source for the location of the 25-inch precipitation contour (18).

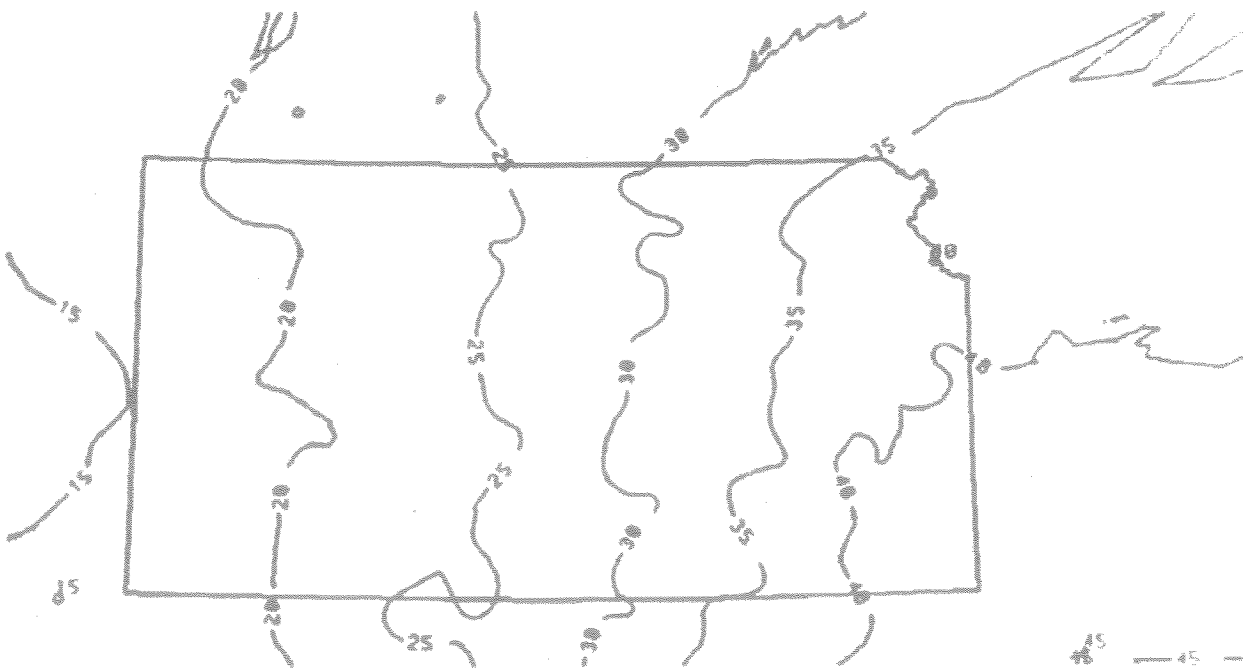

Figure 9. Map of the precipitation contours resulting from applying the UK method with linear drift to the 1990 normals (17). Here, the contour interval is 5 inches. Note the incoherent behavior of the contours around the margins of the map. This is referred to as "edge effect."

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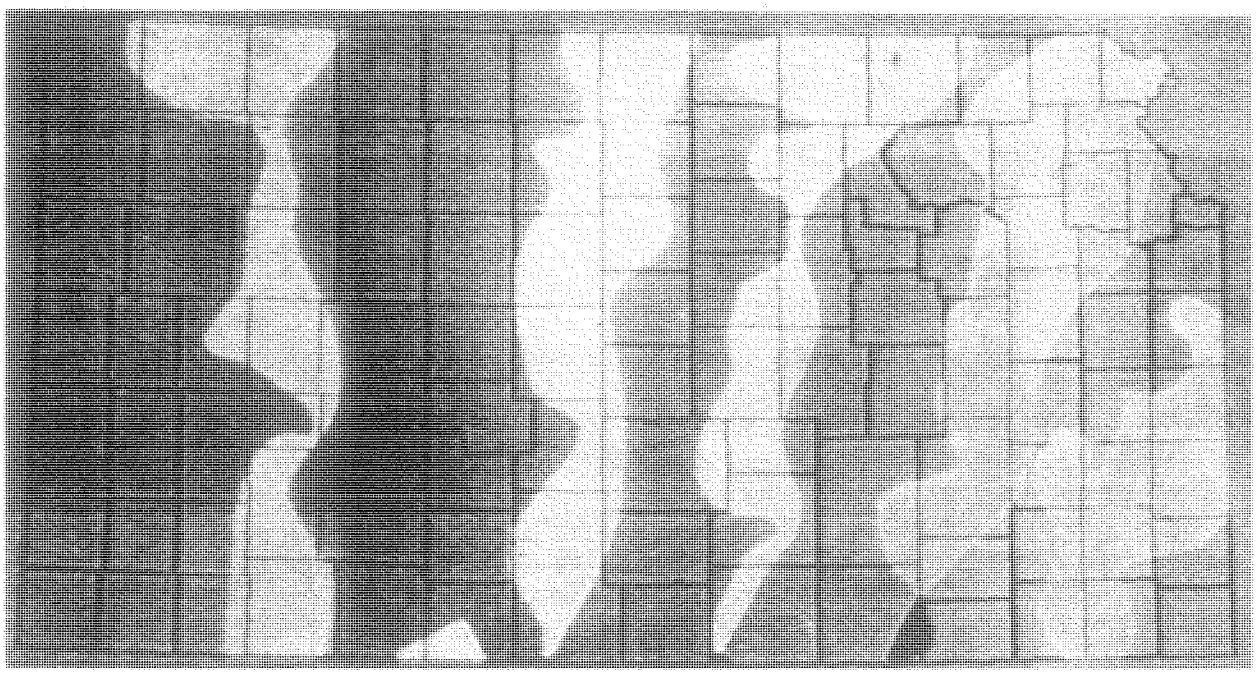

Higher Precipitation in 1990 Data

Lower Precipitation in 1990 Data

Figure 10. This map represents the union (overlay) of the information in Figures 8 and 9. The lighter regions represent areas exhibiting higher precipitation in the 1990 normals (1961 to 1990) than was apparent in the 1960 normals (1931 to 1960). The darker areas show the opposite relationship.

line in Kansas is a clear example of how the use of inappropriate data can have far-reaching effects on policy and management. The regulatory agency, KDHE, chose the wrong map upon which to base its regulatory authority. As a result, numerous potential sites for small municipal solid waste landfills will be considered that are in violation of the letter and intent of the law.

Ultimately, the responsibility for proper use of GIS technology lies in the hands of practitioners. Technical staff performing GIS analysis must be knowledgeable about sources of error and uncertainty and ensure that users of their work are aware of their influence on GIS output.

The problems demonstrated in the Kansas example could have been avoided simply by investigating the appropriateness of the data. Instead, a convenient source was chosen without seeking any other sources of "better" information. Indeed, familiarity with the nature of the data (30-year normals) should have led the policy analyst to select the most current data and not data that are 30 years out of date! An understanding that contour lines represent a generalization of the point precipitation measurements should also have led to the conclusion that locations near the boundary line ought to be monitored for compliance. Both the temporal and spatial characteristics of climate can change, as exemplified by the difference in the 1960 and 1990 normals. The "Dust Bowl" periods of the 1930s and 1950s significantly influenced the 1960 normals (33). As a result, they are not appropriate for this application.

Although a powerful tool, GIS does not hold all the answers. The technical community and policy-makers must work together to ensure its proper use. In reality, no 25-inch precipitation line floats over Kansas. It is merely the interpretation of scientists and policy-makers who select its location. The only way to arrive at a reasonable answer is to gather the best available information and allow all parties to scrutinize it. GIS can be a wonderful tool to do this.

Acknowledgments

The author wishes to express his gratitude to the Kansas Geological Survey for its computing support before, dunng, and after his move to Louisiana. Also, the author wishes to thank Mr. M. Schouest, who made the move both possible and tolerable. Mr. C. Johnson of Johnson Controls in Baton Rouge, Louisiana, produced the excellent presentation graphics. Finally, this work could not have been performed without the Internet.

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Data Quality Issues Affecting GIS Use for Environmental Problem-Solving

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"Abandon hope, all ye who enter here." Dante's quote might well be the advice that experienced geographic information system (GIS) users give to nonusers about to confront data quality issues associated with GIS use. Indeed, after reading this paper, some decision-makers might abandon attempts to use a GIS because of the error associated with it. Others may want to spend an inordinate amount of time and money trying to eliminate all error associated with GIS use. Neither option is prudent.

Data quality is important because it affects how reliable GIS-generated information is in the decision-making process. Too often, the availability of inexpensive digital data overshadows data quality concerns; people frequently use digital data because they are available, not because they have the necessary accuracy.

A GIS can help decision-makers use spatial information more fully than manual methods allow, but sometimes data quality issues cause concern about using GIS-generated outputs. Making environmental decisions without adequate consideration to data quality may lead to an erroneous decision, erode public confidence, or cause an agency to incur liability. This paper attempts to encourage decision-makers to become more aware of data quality issues, including the sources and magnitude of error.

GIS error research has necessarily progressed in a linear fashion, beginning with identifying and classifying sources of error. This paper discusses both inherent (source) error and the error that GIS operations introduce (operational error) during data input, storage, analysis/manipulation, and output (1). Strategies for coping with error and research into error reduction techniques have only recently received attention. Unfortunately, the answers to error management questions such as, "How will the error affect decision-making?" are not clear. The end of this paper covers several error management suggestions and anticipated software improvements designed to reduce errors, however.

Data Quality Concepts and Their Importance

Data quality is a major issue for GIS-generated maps, much more so than it is for paper maps. In part, this is because a GIS can perform operations on spatial data that would be nearly impossible without a GIS because of scale, complexity, and generalization issues (2). Cartographers adjust for these problems when they manually manipulate and instantly combine paper maps by adhering to long-standing cartographic principles, but GIS personnel may not be fully trained in these principles. A GIS enables an analyst, whether trained in cartographic principles or not, to combine or manipulate data in appropriate or in inappropriate, illogical, and erroneous ways. Lack of training coupled with the speed of spatial data manipulation can have serious consequences for an agency whose personnel produce and use GIS-generated maps.

Limited scientific understanding, limited ability to measure data, sampling error, inherent variability, and inadequacy of mathematical representations all contribute to uncertainties associated with spatial data. Uncertainty about spatial data consists of two parts: ignorance and variability. Ignorance means that variables have a "true" value, but it is unknown to us, whereas variability means one value cannot represent the variables.

Data quality defies a simple definition. For this paper, data quality can roughly mean how "good" the data are for a given purpose. People usually think of data quality in terms of error, but the term is broader and encompasses the six components outlined in the next section. Error can mean the difference between the observed values and the "true" value. The "true" value of a variable is usually unknown and unknowable, but for this paper's purposes, "true" could be the known value or the value one would obtain from field measurements (the discussion of data collection tries to dispel the notion that there is one "true" value for many variables, such as soil type in a given area or water temperature in a lake). Imperfect

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equipment or observers and environmental effects cause spatial error. According to Thapa and Bossler (3), errors fall into three categories:

- Gross errors and blunders (people or equipment).
- Systematic errors (which introduce bias).
- Random errors (due to imperfect instruments and observers).

In addition, another view divides spatial error into two different components: accuracy and precision. Accuracy means how close a value is to the "true" value or a known standard (absence of bias). Precision can have two definitions: it can be a measure of dispersion (standard deviation) of observations about a mean, or it can refer to the number of decimal digits used to represent a value (4). In the first definition of precision, a measurement of 6 feet plus or minus 1 foot is more precise than one of 6 feet plus or minus 3 feet. In the second definition, a value of 6.1794 feet is more precise than one of 6.1 feet. Figure 1 provides a graphic explanation of the difference between error, accuracy, and precision.

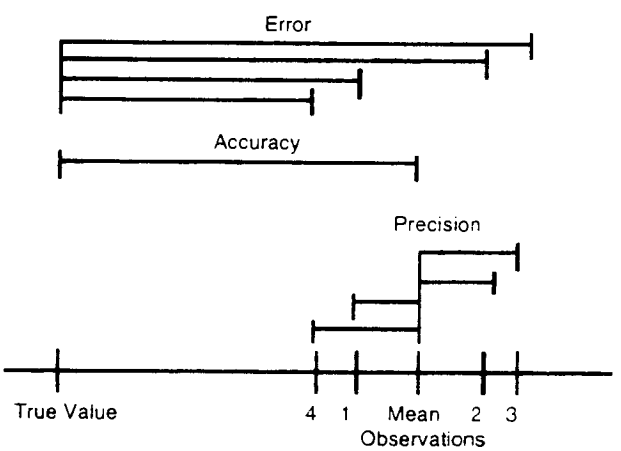

Figure 1. Relationship between error, accuracy, and precision.

Data are not accurate or inaccurate. Instead, data accuracy exists on a continuum, ranging from low to high accuracy. Although people strive for accurate (error-free) data, obtaining 100-percent accurate data is impractical. The list below provides some of the reasons why total accuracy is not obtainable (5):

- Objects to be measured are often vaguely defined.
- Some phenomena are variable in nature.
- Classification schemes are imprecise.
- Measurements are inherently imprecise.
- Gross errors of a nonstatistical nature can occur during measurement.
- Attributes encoded on an ordinal scale (high, medium, low) are approximate.
- Data represent a past state of reality.

Users of geographic data should strive for data that are only as accurate as they need. A variety of factors, of course, can determine need:

- Intended use of the data
- Budget constraints
- Time constraints
- Data storage considerations
- Potential liability

The main barrier to highly accurate data is lack of funds. Nale (6) suggests that rather than abandoning a GIS project because funds are not sufficient to achieve the desired accuracy, an agency should collect data at the desired accuracy from smaller areas, such as areas being developed or redeveloped. Over time, data collection at the desired accuracy can expand to include areas that lacked data due to budgetary constraints. Smith and Honeycutt (7) outline the use of a value of information approach in determining the need for more data (or more accurate data) based on the expected costs and benefits associated with data collection. If the benefits of increased data accuracy are greater than the expected costs, additional funds should be allocated to obtain more accurate data.

The intended use of data affects the type of data, as well as the data quality needed. Beard (8) divides GIS applications into six types (see Table 1). The specific type of data quality one needs (e.g., positional accuracy, attribute accuracy) also varies with the intended application. Analysts with inventory applications such as agricultural production are less concerned about positional accuracy than with an accurate assessment of anticipated crop yields (attribute accuracy). Decision-makers must

Table 1. Types of GIS Applications (8)

Application	Example
Siting	Finding optimal location (fire station, waste site)
Logistic	Movement or distribution through space (emergency response, military movement)
Routing	Optimal movement through a known network (mail, school bus)
Navigation	Way finding; may or may not involve a known network (ground, sea, air)
Inventory	Count and location of objects for a given time (census, tax rolls)
Monitoring/ Analysis	Examining processes over space and time (ecological, zoological, geological, epidemiological studies)

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decide which data quality component is the most important for their use because optimizing all six components can be very expensive (9). An obvious conflict arises when local and state governments must meet multiple application needs simultaneously and thus feel forced to try to optimize several data quality components.

The nature of the decision may also help decision-makers determine the data quality they need. Beard (8) lists several of these factors (see Table 2). A political, high-risk decision requires higher quality data than a nonpolitical, low-risk decision because more public attention focuses on the former decision.

Table 2. Factors That May Affect the Data Quality Needed for Decision-Making (8)

Lower Data Quality Possibly Needed	Higher Data Quality Possibly Needed
Routine	Nonroutine
Nonpolitical	Political
Minimal risk	High risk
Noncontroversial	Controversial
Indefinite	Immediate
Local implication	Global implication

Components of Data Quality

The National Committee for Digital Cartographic Data Standards (9) identifies six components of digital cartographic data quality. This section discusses each of these components:

- Lineage
- Positional accuracy
- Attribute accuracy
- Logical consistency
- Completeness
- Temporal accuracy

Most components of data quality apply to both source and operational error.

Lineage

Because uses and users of data change, those at the national level have noted a recent push to include documentation when disseminating spatial data. Data lineage, also known as metadata or a data dictionary, is data about data. Metadata consists of information about the source data such as:

- Date of collection
- Short definition
- Data type, field length, and format
- Control points used
- Collection method, field notes, and maps
- Data processing steps
- Assessment of the reliability of source data
- Data quality reports

Access to this information can help GIS personnel determine if the data are appropriate for their use, thereby minimizing risks associated with using the wrong data or using data inappropriately. According to Chrisman (10), the only ethical and probably best legal strategy for those who produce spatial data is to reveal more information about the data (metadata) so that users can make informed decisions. Eagan and Ventura's article (11) contains a sample of a generic environmental data lineage report. The U.S. Environmental Protection Agency's (EPA's) new locational data policy requires contractors to estimate data accuracy and provide information about the lineage of the data (12).

Positional Accuracy

Anyone who has used a map has probably come across features that are not located where the map says they should be located and has experienced low positional accuracy. (Undoubtedly, they have also detected features that were not on the map, but that is a different issue.) Positional accuracy, frequently referred to as horizontal error, is how close a location on a map is to its "true" ground position. Features may be located inaccurately on maps for many reasons, including (13):

- Poor field work.
- Distortion of the original paper map (temperature, humidity).
- Poor conversion from raster to vector or vector to raster data.
- Data layers are collected at different times.
- Natural variability in data (tides, vegetation, soil).
- Human-induced changes (altering reservoir water levels).
- Movement of features (due to scale of the map and printing constraints) so they can be easily discerned by the map reader.
- Combining maps with different scales.
- Combining maps with different projection and coordinate systems.
- Different national horizontal datum in source materials.
- Different minimum mapping units.

Positional accuracy has two components: bias and precision. Bias reflects the average positional error of the

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sample points and indicates a systematic discrepancy (e.g., all locations are 7 feet east of where they should be). Estimating precision entails calculating the standard deviation of the dispersion of the positional errors. Usually, root mean square error (RMSE) is reported as the measure of positional accuracy, but it does not distinguish bias from precision (14). RMSE is frequently monitored during digitizing to minimize the introduction of additional positional error into the GIS.

To determine positional accuracy, one must compare the location of spatial data with an independent source of higher accuracy. Federal agencies that collect data and produce maps adhere to National Map Accuracy Standards (NMAS) for positional accuracy. Maps such as United States Geological Survey (USGS) topographic maps that conform to NMAS carry an explicit statement on them. Other groups also have developed standards for large-scale mapping (15).

NMAS for positional accuracy require that not more than 10 percent of well-defined points can be in error by more than one-thirtieth of an inch for maps at a scale of 1:20,000 or larger. For smaller scale maps, not more than 10 percent of well-defined points can be in error by more than one-fiftieth of an inch (16). Thus, less than 10 percent of the well-defined locations on a USGS 1:24,000 map can stand more than 40 feet from their "true" location; the other 90 percent of the well-defined points must stand less than 40 feet from their "true" location. Table 3 shows the acceptable positional accuracy for commonly used maps. Note that as scale decreases from 1:1,200 to 1:100,000, positional accuracy decreases.

Several important issues relate to NMAS. First, not all maps adhere to NMAS, which means their positional accuracy may be lower than NMAS or may be unknown. Second, NMAS do not indicate the location of points in error. Third, 10 percent of the well-defined points can have a positional error greater than the standards allow, but neither the location nor the magnitude of these errors are known. Fourth, NMAS apply to well-defined points; therefore, areas that are not well defined may have even lower positional accuracy. The implication of these errors in location is that users should use caution in making decisions that require high positional accuracy. Positional accuracy issues are particularly troublesome for GIS operations on small-scale maps or when combining large-scale maps (1:1,200) with small-scale maps (1:100,000).

Table 3. NMAS Horizontal (Positional) Accuracy

Scale	1 Inch = x Feet	Horizontal Accuracy +/- Feet
1:1,200	100	3.33
1:2,400	200	6.67
1:4,800	400	13.33
1:12,000	1,000	33.33
1:24,000	2,000	40.00
1:63,360	5,280	105.60
1:100,000	8,333	166.67

Recently, global positioning systems (GPS), which the U.S. military developed, have helped to obtain more accurate feature locations. GPS is not without error, however. The list below notes some of the possible sources of error associated with GPS use, some of which can be controlled while others cannot (17):

- Errors in orbital information.
- Errors in the satellite clocks.
- Errors in the receiver clocks.
- Ionospheric or tropospheric refraction.
- Deliberate degrading of the satellite signal.
- Obstructions that block the signal.
- Reflection of the GPS signal off buildings, water, or metal.
- Human error.

The importance of positional accuracy depends on the intended use of the data. In an urban area, a positional error of 1 foot on a tax map may be unacceptable because 1 foot may be worth millions of dollars. In a rural area, however, tax boundaries mapped within 10 feet of their surveyed location may be accurate enough (6). Somers (18) reports that positional accuracy of 10 to 20 feet may be sufficient for environmental analysis. She says the cost of increasing accuracy to 5 feet could increase the cost of data collection by a factor of 10. The decision-maker must determine the needed positional accuracy.

Attribute Accuracy

Attribute accuracy refers to how well the description of a characteristic of spatial data matches what actually exists on the ground. For some spatial data, the location does not change over time, but the value of the attribute does (e.g., the location of a census tract does not change, but the population within a census tract changes). Attribute accuracy is reported differently for continuous data (i.e., elevation, which has an infinite number of values) or discrete data (i.e., gender, which has a finite number of values).

NMAS exist for elevation contour lines on topographic maps. NMAS for vertical accuracy state that not more than 10 percent of the points tested shall be in error by more than one-half of the contour interval (16). A well-defined point on a USGS topographic map with a 10-foot

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contour interval could vary by 10 feet because the actual elevation could be 5 feet higher or lower than the map indicates. The implications of these errors are similar to the ones for positional accuracy. In addition, errors in elevation are important because small changes in elevation may significantly affect some GIS analysis operations such as the determination of aspect, slope, viewshed, and watershed boundaries.

NMAS do not exist for discrete variables such as land use derived from satellite imagery. Instead, a classification matrix reports attribute accuracy. Field checking or checking a portion of the classified image against a map of higher accuracy determines the accuracy of the land use classification. The result of the comparison is a table from which to calculate overall, producer's, and user's accuracy. Table 4 is an example of a classification accuracy matrix.

Table 4. Example of a Classification Accuracy Matrix (19)

Reference Data ("Ground Truth")
Number of Cells

Classified Data (Satellite Image) Number of Cells	**Forest**	**Water**	**Urban**	**Total**
Forest	28	14	15	57
Water	1	15	5	21
Urban	1	1	20	22
Total	30	30	40	100

Overall Accuracy (sum of the main diagonal)

$$\frac{63}{100} = 63\%$$

Producer's Accuracy (column total)				**User's Accuracy (row total)**			
Forest	=	$\frac{28}{30}$	= 93%	Forest	=	$\frac{28}{57}$	= 49%
Water	=	$\frac{15}{30}$	= 50%	Water	=	$\frac{15}{21}$	= 71%
Urban	=	$\frac{20}{40}$	= 50%	Urban	=	$\frac{20}{22}$	= 91%

Overall accuracy is the percentage of correctly classified cells calculated as the sum of the main diagonal (19). Producer's accuracy is the total number of correct pixels in a category divided by the total number of pixels of that category as derived from the reference data (column total). It corresponds to how well the person classifying the image (the "producer") can correctly classify or map an area on the earth. In this example, the producer most accurately classified forested land (93 percent). User's accuracy describes the probability that a sample from the classified area actually represents that category on the ground. The map "user" is concerned about the map's reliability. In this example, the most accurately classified land use from the user's perspective is urban (91 percent).

The significance of overall, producer's, and user's accuracy depends on the intended use of the data. As an example, Chrisman (20) says that the error in distinguishing wetland from pasture may not matter to someone estimating open space, but the difference is critical if the person is estimating the amount of wildlife habitat available. Story and Congalton (19) provide an example of how to interpret a classification matrix. A forester looks at the classification matrix and sees that forest classification is 93 percent accurate (producer's accuracy); therefore, the analyst did not identify only 7 percent of the forest on the ground. Once the forester field checks the supposed forested area, she finds that only 49 percent (28 cells) of the sites mapped as forest are actually forest; the rest are water (14 cells) or urban (15 cells) areas.

A report of overall, producer's, and user's accuracy can help decision-makers determine the appropriateness of the classified image for their use by identifying potential errors in classification. This can help direct field work, which can improve the classification of the image and perhaps subsequent images. Because GIS analysis frequently uses land use, decision-makers need to know that significant variability can result when several analysts classify the same image. Bell and Pucherelli (21) found that consistency in classification can improve by having one person classify the entire image. McGwire (22) even found significant differences between analysts in unsupervised classification of Landsat imagery. Computers primarily perform unsupervised classification, which implies that different analysts would classify the same image in the same way.

Logical Consistency

Logical consistency focuses on flaws in the logical relationships among data elements. For example, a vector GIS should label all polygons with only one label per polygon, and all polygons should be closed. Logical inconsistency can also occur by collecting data layers at different times or from different scale maps with different positional accuracies. For example, the edge of a lake on the hydrology data layer should coincide with the edge of land in the land use data layer. If data on the lake were collected during a wet year rather than a dry year, the lake's volume would be higher than normal, affecting its location on the map. If land use data for the same area were collected during a dry year, the boundary of the lake on the two layers would not be the same.

Logical inconsistencies usually do not appear until the two maps are overlaid and the boundaries do not coincide (see Figure 2). The user must determine the "correct" location of the feature that appears misaligned on one or more data layers. The inconsistency between the

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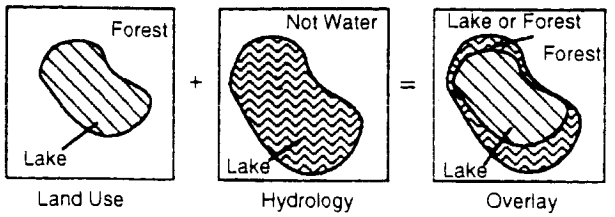

Figure 2. Logical inconsistency in lake and forest location.

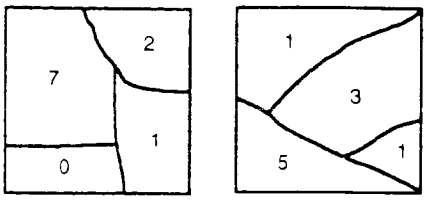

Figure 3. Modifiable area unit. (Number of units is constant; location of units changes.)

location of the two layers resolves through a process called conflation. All maps are adjusted so that the feature on each data layer lines up with the same feature on the base map.

Completeness

Completeness focuses on the adequacy of data collection procedures. Robinson and Frank (5) discuss two kinds of uncertainty associated with collecting spatial data that can lead to error. One type of uncertainty is the inability to measure or predict an inherently exact characteristic or event with certainty. Examples of this are blunders in data collection or measurement error, neither of which can be accurately predicted. The other kind of uncertainty is associated with concepts that are inherently ambiguous. Crisp data sets, such as property boundaries, have little ambiguity; the only issue related to error is the positional accuracy in measuring the boundary. Because land use data are not crisp data sets, the challenge is to accurately represent an inherently inexact concept.

Although we know spatial data are variable, our classification systems generally ignore the second type of uncertainty. Analysts map data as though all variables had exact boundaries and all polygons consisted of homogeneous data. Burrough (4) reports that spatial variation of natural phenomena is "not just a local noise function or inaccuracy that can be removed by collecting more data or by increasing the precision of measurement, but is often a fundamental aspect of nature that occurs at all scales. . . ."

Mapping spatial data is a function of how humans aggregate and disaggregate data either in space, categories, quantities, or time; spatial data seldom exist in nature the way maps depict them (23). Data and relationships between data are sensitive to the scale and the zoning system in which the data are reported (24, 25). The modifiable area unit problem occurs because an analyst can recombine a given set of units or zones into the same total number of units producing very different results (see Figure 3).

The scale problem occurs because an analyst can combine a set of small units into a smaller number of larger units, which can change the inferences that can be made from the data. In Figure 4, the area containing the highest values changes from the southwest corner in the first picture to the northern half in the second picture. For example, water quality data are scale-dependent because they vary based on the size and location of the collection area (e.g., adjacent to a point source discharge, a stream segment, the entire river, or the lake the river discharges into).

Kennedy (25) reports on a similar problem known as the small number problem. This problem occurs when calculations use a percentage, ratio, or rate for a geographic area for which the population of interest (denominator) is sparse or the numerator is a rare event (1 case of cancer per 1 million people). GIS-generated maps may highlight a statistically insignificant change in rare events. Small, random fluctuations in the numerator may cause large fluctuations in the resulting percentage, ratio, or rate. If policy-makers use these maps, priorities for public health policy may change because of the erroneous belief that an area is experiencing more unwanted rare events.

Data can be collected using a tag- or count-based system, which affects their usefulness. The tag approach categorizes items based on the dominant or average attribute and is ideal for planners who want only one value for each area. For example, each polygon in a county soil survey is tagged with one soil type. According to soil taxonomy rules, however, only about 35 percent of a delimited area on a soil survey must match its classification, and up to 10 percent may be a radically different soil (26). Although the text in the soil survey sets limits on data accuracy by listing major impurities found with each soil type, the GIS seldom carries that information because analysts only digitize soil boundaries and label data with the dominant attribute. This

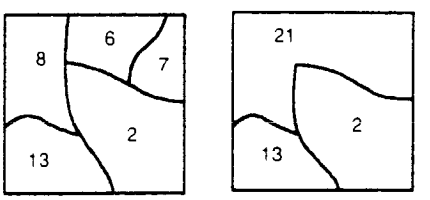

Figure 4. Scale problem (number of units changes).

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leads to the depiction of apparently homogeneous soil units although the text specifies that the data are not homogeneous (27).

Some soil or land cover phenomena, even though present in small quantities and thus not mapped, may have great significance for hydrologic models, which makes the tag approach to data collection troublesome. Data collected using the count system allow the analyst to tabulate the frequency of occurrence or areal extent of a particular phenomenon. Environmental modelers prefer count data but are usually forced to use tag data, which can introduce error into their models (26). The new digital soils databases, STATSGO and SSURGO, are collected and depicted using a count format, which will help experienced analysts use the data more fully. Figure 5 shows the difference between tag and count methods of data collection.

Data are seldom complete because analysts use classification rules to indicate how homogeneous an area must be before it is classified a particular way (e.g., more than 50 percent, more than 75 percent). Another decision an analyst must make is where to draw the boundary between two different areas; it is seldom clear where a forest leaves off and a rural development begins. Analysts must also decide how or if to show inclusions (e.g., a forested area in the middle of agricultural land uses).

Temporal Accuracy

Collecting data at different times introduces error because the variable may have changed since data collection. The effect of time, reported as the date of the source material, depends on the intended use of the data. Some natural resource data have daily, weekly, seasonal, or annual cycles that are important to consider. For example, obtaining land use data from remotely sensed imagery in November for North Dakota produces a very different land use map than data analysts obtain during the July growing season.

In addition, demographic and land use information changes quickly in a rapidly urbanizing area. Data collected at several times can produce logical inconsistency between data layers, forcing the analyst to adjust the location of features to coincide with the base map. Another problem with collecting data at different times

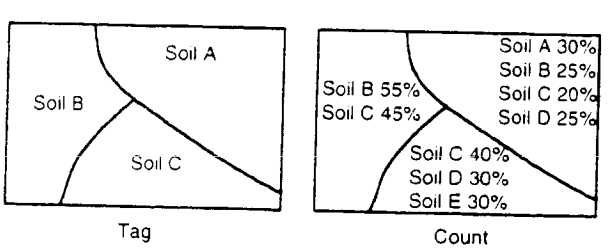

Figure 5. Tag and count methods of data collection.

is that data may be collected using different standards, which may not be apparent to the user (4).

Source Errors in a GIS

Source (or inherent) error derives from errors in data collection. The amount of error present in collected data is a function of the assumptions, methods, and procedures used to create the source map (28). Primary data refers to data collected from field sampling or remote sensing. Causes of the errors associated with this data are (3, 4, 8, 14, 29):

- Environmental conditions (e.g., temperature, humidity).
- Sampling system (e.g., incomplete or biased data collection).
- Time constraints.
- Map projection.
- Map construction techniques.
- Map design specifications.
- Symbolization of data.
- Natural variability.
- Imprecision due to vagueness (e.g., classifying a forest).
- Measurement error from unreliable, inaccurate, or biased observers.
- Measurement error from unreliable, inaccurate, or biased equipment.
- Lab errors (e.g., reproducibility between lab procedures and between labs).

The process of converting primary data to secondary data (usually a map) introduces additional error. Many of the data layers that a GIS analyst acquires are secondary data. Some of the errors associated with map-making are (3):

- Error in plotting control points.
- Compilation error.
- Error introduced in drawing.
- Error due to map generalization.
- Error in map reproduction.
- Error in color registration.
- Deformation of the material (temperature, humidity).
- Error introduced due to using a uniform scale.
- Uncertainty in the definition of a feature (boundary between two land uses).
- Error due to feature exaggeration.
- Error in digitization or scanning.

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Converting paper maps to digital data for entry into a GIS (tertiary data) introduces still more error (the errors generated from converting paper maps into a digital format are discussed in the section on input error), in part because the purpose for which the data was collected differs from the intended use of the data.

Many types of error are associated with data collection:

- Data for the entire area may be incomplete.
- Data may be collected and mapped at inappropriate scales.
- Data may not be relevant for the intended application.
- Data may not be accessible because use is restricted.
- Resolution of the data may not be sufficient.
- Density of observations may not be sufficient.

The following discussion explains these types of errors.

Data for the Entire Area May Be Incomplete

An incomplete data record may be due to mechanical problems that interrupt recording devices, cloud cover or other types of interference, or financial constraints. Possible solutions to this problem include collecting additional data for the incomplete area, using information from a similar area, generalizing existing large-scale maps to match the less detailed data needed, or converting existing small-scale maps to large-scale maps to obtain data at the desired scale. Collecting additional data may not be a feasible solution because of time or money constraints. Extrapolating data from the surrogate area to the desired area can cause problems because the areas are not identical and the scale, accuracy, or resolution of the surrogate area data may be inappropriate for the intended use. The section on analysis/manipulation of data within a GIS covers the effect of generalization on data quality as well as the effect of converting small-scale maps to large-scale maps.

Data May Be Collected and Mapped at a Scale That Is Inappropriate for the Application

A variety of guidelines suggest the appropriate map scale to use for various applications (see Table 5). Also, some maps and digital databases suggest the type of application for which they are appropriate (e.g., the STATSGO digital soil database is suitable for state and regional planning, whereas SSURGO is suitable for local level planning). Tosta (30) cites an example of combining wetland data with parcel boundaries to determine ownership of the land containing a wetland. If wetland mapping was done to plus or minus 100 feet positional accuracy and parcels are 40 feet wide, then the scale of the wetland map is inappropriate for determining if a wetland is located on a specific parcel.

Table 5. Relationship Between Map Scale and Map Use (6)

Map Scale	Map Use
1:600 or larger	Engineering design
1:720 to 1:1,200	Engineering planning
1:2,400 to 1:4,800	General planning
1:6,000 and smaller	Regional planning

Identifying the optimal scale of the necessary data is crucial because at some point, the cost of collection and storage exceeds the benefits of increasing the map scale. Lewis Carroll (1893) summed up the quest for data mapped at an ever larger scale and the problems associated with large-scale maps:

> "What do you consider the *largest* map that would be really useful?"
>
> "About six inches to the mile."
>
> "Only *six inches!*" exclaimed Mein Herr. "We very soon got to six *yards* to the mile. Then we tried a *hundred* yards to the mile. And then came the grandest idea of all! We actually made a map of the country, on the scale of *a mile to the mile!*"
>
> "Have you used it much?" I enquired.
>
> "It has never been spread out, yet," said Mein Herr. "The farmers objected: they said it would cover the whole country, and shut out the sunlight! So now we use the country itself, as its own map, and I assure you it does nearly as well."

Data Collected May Not Be Relevant for the Intended Application

Frequently, using surrogate data is quicker or cheaper than collecting needed data (e.g., Landsat imagery rather than data field collection used to determine land use) (4). The accuracy and classification scheme used in collecting the data depends on the intended use of the data, which may not coincide with the analyst's purpose. For instance, soil maps were developed to aid farmers in determining what crops they should plant and for estimating crop yield. Soil maps, however, see wide use for very different purposes (e.g., hydrologic and other environmental models). In addition, STORET data, collected at points, are typically extrapolated to represent water quality in an entire stream stretch.

Data May Not Be Accessible Because Use Is Restricted

An example of restricted data is Census data on individual households. An agency may not want to release data that reveal the location of endangered species. Another

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example is that people may not even want the information mapped. For example, some cavers do not want to reveal the location of caves to the U.S. Forest Service, which is charged under the federal Cave Resources Protection Act with protecting caves, because they think the best way to protect the caves is to not map them (31). The National Park Service is putting the location of petroglyphs in the Petroglyph National Monument into a GIS. Making their location known to the public, however, is troublesome because this may, in fact, encourage their vandalism (32). Other problems in obtaining data include difficulty in acquisition even if access is not restricted, expensive collection or input, or unsuitable format (4, 14).

Resolution of the Available Data May Not Be Sufficient

Spatial resolution is the minimum distance needed between two objects for the equipment to record the objects as two entities; that is, resolution is the smallest unit a map represents. To obtain an approximation of a map's resolution, divide the denominator of the map scale by 2,000 to get resolution in meters; for instance, a 1:24,000-scale map has a resolution of approximately 12 meters (33).

Resolution relates to accuracy in that different map scales conform to different accuracy standards. Two air photos shot from the same camera at the same distance above the ground have the same scale. If one photo has finer grain film, however, smaller details are evident on it, and this photo produces a map with higher resolution (34). According to Csillag (33), analysts cannot simultaneously optimize attribute accuracy and spatial resolution. As spatial resolution increases, attribute complexity increases (35). Also, the finer the spatial resolution, the greater the probability that random error significantly affects a data value.

Resolution of the data is not necessarily the same as the size of a raster cell in a database. Statistical sampling theory suggests using a raster cell size that is half the length (one-fourth of the area) of the smallest feature an analyst wishes to record. Raster data have a fixed spatial resolution that depends on the size of the cell employed, but a GIS analyst can divide or aggregate cells to achieve a different cell size. Frequently, an analyst transforms data collected at one level of resolution to a higher level of resolution than existed in the original source material. According to Everett and Simonett (23), "Geographic analysis, however, can be no better than that of the smallest bit of data which the system is capable of detecting." Vector data are limited by the resolution of input/output devices, limits on data storage, and the accuracy of the digitized location for individual points (36). The spatial resolution of the database and the processes that operate on it should be reduced to a level consistent with the data's accuracy.

The spatial resolution needed depends on the intended use of the data, cost, and data storage considerations. As resolution increases, so does the cost of collection and storage. Resolution sufficient to detect an object means that an analyst can reveal the presence of something. Identification, the ability to identify the object or feature, requires three times the spatial resolution of detection. Analysis, a finer level of identification, requires 10 to 100 times the resolution that identification needs (23). Increasing resolution increases the amount of data for storage, with storage requirements increasing by the square of the resolution of the data. For example, if the resolution of the data needs to change from 10-meter to 1-meter pixels, file size increases by 10^2 or 100 times (14).

Density of Observations May Be Insufficient

The density of observations serves as a general indicator of data reliability (4). Users need to know if sampling was done at the optimum density to resolve the pattern. Burrough determined that boulder clay in The Netherlands could be resolved by sampling at 20-meter intervals or less, whereas coversand showed little variation in sampling from 20- to 200-meter intervals.

Some strategies for reducing data collection errors are to:

- Adhere to professional standards
- Allocate enough time and money
- Use a rigorous sampling design
- Standardize data collection procedures
- Document data collection procedures
- Calibrate data collection instruments
- Use more accurate instruments
- Perform blunder checks to detect gross errors

Documenting data collection procedures and distributing them along with data allows potential users to determine if the data are suitable for their purposes. By not documenting procedures, errors in the source material are essentially "lost" by inputting the data to a GIS, and the errors become largely undetectable in subsequent GIS procedures. The result is that agencies that make decisions based on the GIS-generated map assume the source data are accurate, only to discover later that the map contains substantial errors in part due to errors in the source material.

Operational Errors in a GIS

Data input, storage, analysis/manipulation, and output can introduce operational errors. Digital maps, unlike

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paper maps, can accumulate new operational errors through GIS operations (8). Even if the input data were totally error-free, which the last section demonstrated is not the case, GIS operations can produce positional and attribute errors. The GIS operation itself determines to a large extent the types of errors that result.

Input Errors

The process of inputting spatial and attribute data can introduce error. The major sources of input error are manual entry of attribute features and scanning or digitizing spatial features. Manual entry errors include incomplete entry of attribute data, entering the wrong attribute data, or entering the right attribute data at the wrong location. Digitizing errors originate from equipment, personnel, or the source material (see Table 6).

Digitizing errors, such as under- and overshoot of lines and polygons that are not closed, can introduce error (see Figure 6). GIS software can "snap" lines together that really do not connect. Depending on the tolerance selected, this can result in the movement of both lines, which can decrease the accuracy of the resultant map.

Table 6. Types of Digitizing Errors (4, 14, 37)

Personnel Errors
Changes in the origin
Incorrect registration of the map on the digitizing table
Creation of over- and undershoots
Creation of polygons that are not closed
Incomplete spatial data when data are not entered
Duplication of spatial data when lines are digitized twice
Line-following error (inability to trace map lines perfectly with the cursor)
Line-sampling error (selection of points used to represent the map)
Physiological error (involuntary muscle spasms)
Equipment Errors
Digitizing table (center has higher positional accuracy than the edges)
Resolution of the digitizer
Differential accuracy depending on cursor orientation
Errors in Source Material
Distortion because source maps have not been scale-corrected
Distortion due to changes in temperature and humidity
Necessity of digitizing sharp boundary lines when they are gradual transitions
Width of map boundaries (0.4 mm) digitized with a 0.02-mm accuracy digitizer

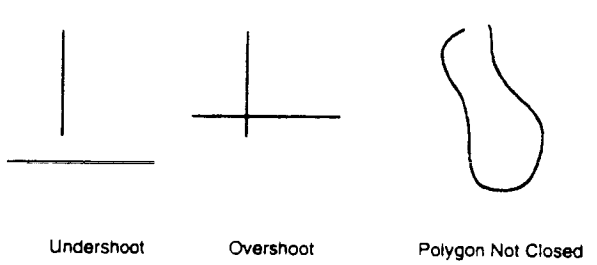

Figure 6. Common digitizing errors.

Despite the long list of personnel errors associated with digitizing, a good operator probably contributes the least error in the entire digitizing process (38). Giovachino discusses methods that can help determine equipment accuracy, including checking the repeatability, stability, and effect of cursor rotation. Digitizing accuracy varies based on the width, complexity, and density of the feature being digitized but typically varies from 0.01 to 0.003 (3).

One problem with digitized data is that the data can imply a false sense of precision. Boundaries on paper maps are frequently 0.4 mm wide but are digitized with 0.02-mm accuracy. The result is that the lines are stored with 0.02-mm accuracy, implying a level of precision that far exceeds the original data.

Minimizing digitizing errors is important because the errors can affect subsequent GIS analysis. Campbell and Mortenson (39) provide a list of procedures they used to reduce errors associated with digitizing and labeling:

- Use log sheets to ensure consistency and accountability, and to provide documentation.
- Check for completeness in digitizing all lines and polygons.
- Check for complete and accurate polygon labeling.
- Set an acceptable RMSE term for digitizing (usually 0.003).
- Always overshoot rather than undershoot when digitizing.
- Overlay a plot of the digitized data with the source map to check lines and polygons. If light passes between the digitized line segment and source map, redigitize it.
- Check digitized work immediately to provide feedback to the digitizer operator and to help identify and correct systematic errors.
- Limit digitizing to less than 4 hours a day.
- Involve people in doing GIS-related jobs other than digitizing to decrease turnover and increase the level of experience.

Storage Errors

Data storage in a GIS usually involves two main types of errors. First, many GIS systems have insufficient numerical precision, which can introduce error due to rounding. Integers are stored as 16 or 32 bits, which have four significant figures. Real numbers are stored as floating point numbers either in single precision (32 bit,

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7 significant figures) or double precision (64 bit, 15 or 16 significant figures). If the data in a GIS range from fractions of a meter to full UTM coordinates, typical 32-bit GIS systems cannot store all the numbers. Using double precision (64 bits) reduces this problem but increases storage requirements.

Second, GIS processing and storage usually ignore significant digits (data precision). As a result, the precision of GIS processing frequently exceeds the accuracy of the data (40). When a GIS converts a temperature recorded and entered as 70 degrees Fahrenheit (nearest degree) to centigrade, the GIS stores the temperature as 21.111 degrees rather than 21 degrees, which the significant figures in the original temperature measurement would dictate. Using the accuracy of the data, not the precision of floating point arithmetic, partially resolves this but requires the user to make a special effort because the GIS does not automatically track significant figures.

Analysis/Manipulation Errors

GIS analysis/manipulation functions, designed to transform or combine data sets, also can introduce errors. These errors originate from the measurement scale used or during data conversion (vector to raster and raster to vector), map overlay, generalization, converting small-scale to large-scale maps, slope, viewshed, and other analysis functions. One of the biggest problems associated with GIS use is that data in digital form are subject to different uses than data in paper form because the user has access to multiple data layers.

Measurement Scale

Four measurement scales can depict spatial data: nominal, ordinal, interval, or ratio scales. A name or letter describes nominal data (e.g., land use type, hydrologic soil group C). Performing mathematical operations such as addition and subtraction on nominal data is meaningless. Ordinal or ranked data have an order to them such as low, medium, and high. Interval data have a known distance between the intervals such as 0, 1 to 5, 6 to 9, more than 9. Ratio data are similar to interval data except ratio data have a meaningful zero (e.g., temperature on the Kelvin scale).

Often during GIS operations, analysts convert interval or ratio data into nominal data (e.g., low slope is 0 to 3 percent, medium slope is 4 to 10 percent), resulting in a loss of information. Analysts should preserve the original slope values in the GIS in case the user later wants to modify the classification scheme. Robinson and Frank (5) describe the tradeoff between information content and the meaning that can be derived from it, which partly helps explain why interval data are frequently converted to nominal data. The authors identify a continuum progressing from nominal data at one end that is highly subjective, has low information content, and high meaning (low slope means something to the average user) to ratio data that has low subjectivity, high information content, and low meaning (a slope of 7 percent may not mean much to the average user).

Data Conversion

Errors can occur in converting a vector map to a raster map or a raster map to a vector map. For instance, remotely sensed data are collected using a raster-based system. Using a vector GIS, however, requires conversion from raster to vector data. The size of the error depends on the conversion algorithm, complexity of features, and grid cell size and orientation (13).

A line on a vector map converted to a raster map has lower accuracy in the raster representation because vector data structures store data more accurately than raster ones. When polygons in a vector GIS are converted to a raster GIS, the coding rule usually used assigns the value that covers the largest area within the cell of a categorical map to the entire cell (see Figure 7). For example, when placing a grid over a vector map with an urban land polygon adjacent to an agricultural polygon, the cell placement can include part of both polygons. If the resultant cell comprises 51 percent urban and 49 percent agricultural land, the cell is assigned 100 percent urban. Converting a numerical map between raster and vector systems requires spatial interpolation procedures. GIS software packages use different interpolation methods that can produce a different output even when using the same input data.

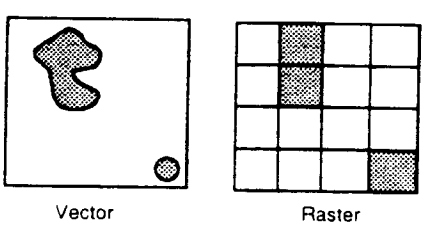

Figure 7. Polygon conversion from vector to raster data.

Map Overlay

Map overlay, used extensively in planning and natural resource management, is the combining of two or more data layers to create new information. In a vector GIS, slivers or spurious polygons can result from overlaying two data layers to produce a new map (slivers cannot be formed in a raster-based GIS). When combining the data layers, lines do not coincide, resulting in the creation of a new polygon or sliver that did not exist on either layer (see Figure 8). Unfortunately, as accuracy in digitizing increases, so does the number of slivers (41). Positional error in the boundaries can occur because of

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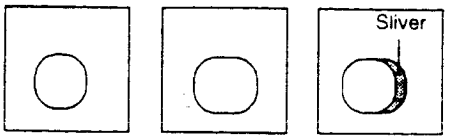

Figure 8. Sliver example.

mistakes in measuring or converting the data to digital form, incremental expansion or recession of a real world boundary over time, or the fact that certain boundaries are difficult to determine and thus are generalized differently (42).

The number of map layers, accuracy of each map layer, and the coincidence of errors at the same position from several map layers all determine the accuracy of the map overlay procedures (43). Using probability theory, Newcomer and Szajgin determined that the highest accuracy to expect from a map overlay is equal to the accuracy of the least accurate map layer. The lowest accuracy in map overlay occurs when errors in each map occur at unique points.

In the quest for more accurate results, GIS modelers have increased the complexity of their models and therefore have increased the number of data layers needed. Guptill (44) states, "Conventional wisdom would say that as you add more data to the solution of a problem, the likelihood of getting an accurate solution increases. However, if each additional data layer degrades the quality of the combined data set, and hence the accuracy of the solution, then additional data sets may be counterproductive."

Generalization

Monmonier (45) provides an extensive discussion of geometric and content generalization procedures used in map-making. Table 7 lists common types of generalization. Generalizing data on a map helps to focus the user's attention on one or two types of information and to filter out irrelevant details. Generalizing is performed by reducing the scale of the data; a 1:24,000-scale map can be generalized to a 1:100,000-scale map so that all data layers have the same scale. With generalizing, areas on a large-scale map become point or line features on a small-scale map (35). Obtaining some measurements from small-scale maps, however, requires caution. For example, a map may depict a 40-foot wide road as a single line one-fiftieth of an inch wide. On a 1:100,000 map, one-fiftieth of an inch translates into a 160-foot wide road—four times the actual width of the road.

Several studies have pointed to errors that can result from generalization. Wehde (46) compared soil maps generated from 0.017-acre grid cells and 11 progressively increasing grid cell sizes. He found that as grid cell size increased, map accuracy decreased. More recently, Stoms (47) found that generalizing a habitat map from 1 to 25, 100, and 500 hectares decreased the number of habitat types and the number of species predicted.

Table 7. Generalization Operations (45)

Geometric Generalization
Generalizing a Line
Simplification
Displacement
Smoothing
Enhancement
Selection
Generalizing a Point
Selection
Displacement
Graphic association
Abbreviation
Aggregation
Area conversion
Generalizing an Area
Selection
Simplification
Displacement
Smoothing
Enhancement
Aggregation
Dissolution
Segmentation
Point conversion
Line conversion
Content Generalization
Selection
Classification

Transforming Small-Scale Maps to Large-Scale Maps

Converting small-scale maps (1:250,000) to large-scale maps (1:24,000) is advisable only if the analyst fully appreciates the effect of this procedure on map quality. Data mapped at a small scale are subject to different accuracy standards than data mapped at a large scale. Connin (48) reports, "Problems with accuracy arise when positions are reported to decimal parts of a foot or meter, but the method of data capture may cause the positional error to be as much as hundreds of feet or meters." Yet when converting the data from small- to large-scale, the data appear to have the accuracy of the large-scale map. Theoretically, data should not be transformed and used at a scale larger than the scale of the document from which the data are derived (3).

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Slope and Viewshed

GIS software packages use a variety of algorithms to calculate slope and viewsheds and can produce very different results. Algorithms are an unambiguous set of rules or a finite sequence of operations used to carry out a procedure. Smith, Prisley, and Weih (49) used six different GIS algorithms to determine slope on 5,905 acres of land in order to calculate the amount of land deemed unsuitable for timber harvesting. They found that unsuitable land varied from 175 to 1,735 acres, indicating that different algorithms produce very different results. Felleman and Griffin (50) found that GIS packages with different algorithms generate alternate viewsheds (the area that can be seen from a point).

Output Errors

A variety of errors are associated with data output:

- Output devices create error.
- Paper shrinks and swells.
- Line implies certainty that may not exist because boundaries are gradual.
- A cell or polygon implies homogeneity.
- Scale can be modified to imply higher accuracy than exists in the source data.
- Precision can be modified to imply higher precision than exists in the source data.
- Depiction of symbols and colors may not follow convention.

An important problem associated with GIS-generated maps is that users make informal assessments about data quality, partially based on how they perceive the quality of the output. A hand-drawn map connotes a lower level of accuracy than a five-color, GIS-produced map complete with scale and agency logo. Another problem with output is that distinguishing highly accurate data from less accurate data is impossible on a GIS-generated map. Users want the output from a GIS to look like maps they usually see, perpetuating the notion that lines mark exact boundaries and that polygons or cells are homogeneous. Maps that federal mapping agencies produce frequently follow NMAS, but GIS-generated maps seldom adhere to published map accuracy standards. An agency could require that GIS map products meet NMAS, which would establish and maintain data standards from data collection to output.

A pen stroke of one-fiftieth of an inch on an output device translates to an error of 40 feet on the ground for a 1:24,000-scale map (6). Small changes in paper maps due to changes in temperature and humidity can represent several feet on the ground. As previously noted, analysts can modify the scale of GIS maps to whatever they desire. The basic rule of informational integrity is that the implied precision of data output should not exceed the precision (spatial, temporal, or mathematical) of the least precise input variable (26).

GIS-generated maps probably do not differ significantly from paper maps in their implication that lines and polygons on the map represent certainty and homogeneity. GIS-generated maps, however, may not depict standard symbols, sizes, shapes, colors, and orientation. For example, paper geological maps use dashed lines to show inferred, rather than actual, field collected data, but geological maps in a GIS may not follow the same convention (27). Cartographers conventionally use blue lines to indicate water, but a GIS map-maker can show water as red rather than blue.

Even more troublesome are the color schemes that some analysts use in depicting model output. Analysts often give little thought to assigning the colors to model results depicted as ordinal rankings. For example, areas of high erosion might be blue, medium erosion might be red, and low erosion might be green. This selection of colors ignores the intuitive meaning that people assign to colors. It has been suggested that the color ordering used in stop lights might provide a better option. In that case, areas of high erosion would be red, medium erosion would be yellow, and low erosion would be green.

Error Reduction Techniques

Although GIS users and researchers develop error reduction strategies, ultimately users must rely on GIS software developers to implement new error reduction techniques in GIS packages. Error reduction techniques range from simple software warnings to prohibiting a user from performing selected GIS procedures. Dutton (51) predicts that future GIS programs will automate data manipulation (i.e., size, format, and placement of feature labels on maps) in keeping with standard cartographic principles. Dutton (51) and Beard (8) also predict that future GIS packages will enforce metadata-based constraints such as operations that are illegal or illogical (e.g., determining the average value of nominal data such as land use), or are inadvisable (e.g., overlaying maps with widely different scales).

Another change Dutton anticipates is that software vendors will include information in manuals that explains how executing a specific command may affect the database. Graphic techniques to depict error are being developed for nonexpert users while experts tend to use spatial statistics. Felleman (52) and Berry (53) present an interesting graphic portrayal of an error map that may indicate the future of error maps. Additional research must determine what effect errors will have on decision-making.

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Error Management

Ultimately, the decision-maker must determine what to do with the information in this paper. A decision-maker has a variety of possible courses of action, ranging from prudent steps that attempt to minimize error and the effect it has on decisions, to other less useful options. Possible actions are to:

- Abandon use of a GIS.
- Ignore the error associated with GIS use.
- Attempt to collect "error-free" data.
- Determine if the data are accurate enough for the intended purpose.
- Develop and use data quality procedures.
- Obtain and use an error report with GIS-generated output.
- Ask that GIS-generated maps show potential errors.
- Continually educate users about the appropriate use of spatial data.

First, the decision-maker could abandon any attempt to use a GIS because of the errors associated with its use. At times, this may be the appropriate strategy, but this approach ignores the potential benefits associated with GIS use.

Second, the decision-maker could ignore the error associated with GIS use and continue to use the GIS for decision-making. This type of "head in the sand" approach is not advisable because of the potential liability associated with making decisions based on inaccurate data.

Third, the decision-maker could engage in an expensive and time-consuming effort to collect highly accurate error in hopes that error becomes a nonissue. Depending on the intended use of the data, the cost of collecting more accurate data may exceed the benefit.

Fourth, the decision-maker could assess whether the information available is accurate enough for the intended purpose. If data quality is too low, the decision-maker may opt to collect new data at the desired quality If collecting additional data is not possible, the decision-maker can explore what types of decisions are possible given the attainable data quality. For instance, Hunter and Goodchild (54) found that the data they were using were suitable only for initial screening rather than for regulatory and land-purchasing decisions.

Fifth, procedures to ensure high quality data could be developed and used in the data collection, input, and manipulation stages of building a GIS database.

Sixth, the decision-maker could require a quantitative or at least a qualitative report on the sources, magnitude, and effects of errors. The absence of an error report does not mean the map is error-free (36). Dutton (51) predicts that in the near future users of geographic data will demand error reports, confidence limits, and sensitivity analyses with GIS-generated output.

Seventh, the decision-maker could ask for GIS-generated maps that adequately portray the error in the final map. For example, areas where the uncertainty is high could appear in red on maps. Another option is to place a buffer around lines to indicate the relative positional accuracy of a line or to show transition zones. Finally, an analyst can present the output in ways other than a dichotomous yes or no; instead, the analyst may use yes, maybe, or no depictions or even more gradations.

Finally, Beard (8) introduced the concept of directing efforts toward educating users about use error. She defines use error as the misinterpretation of maps or misapplication of maps to tasks for which they are not appropriate. "We can't assume that GIS will automatically be less susceptible to misuse than traditional maps, and it may, in fact, exacerbate the problem by expanding access to mapped information." Beard argues that money directed to reducing source and operational error, while important, may not matter if use error is large.

Conclusions

GIS is a powerful tool for analyzing spatial data. Everyone who uses GIS-generated output, however, must be aware of source errors and operational errors introduced during data input, storage, analysis/manipulation, and output. Increased awareness of the sources and magnitude of error can help decision-makers determine if data are appropriate for their use. Decision-makers cannot leave data quality concerns to GIS analysts because efforts to improve data quality are not without cost, and the decision-makers typically control funding.

Decision-makers must not get caught up in the glamour of the spatial analyses and outputs that a GIS can produce. These attributes may lead decision-makers to ignore issues associated with uncertainty, error, accuracy, and precision. Inexpensive digital data can make analysts and decision-makers ignore data quality. If subsequent management decisions are made based on poor quality data, the resultant decisions may turn out wrong. This would give decision-makers a jaded view of the usefulness of GIS. An adequate understanding of data quality issues can help decision-makers ask the right questions of analysts and avoid making decisions that are inappropriate given the data quality.

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You Can't Do That With These Data! Or: Uses and Abuses of Tap Water Monitoring Analyses

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Introduction

Linkage between human health and drinking water quality has been an area of interest in the United States for many years. Over the past approximately 10 years, drinking water monitoring requirements have expanded rapidly under the Safe Drinking Water Act (SDWA). Growing public and governmental interest in this environmental area makes the aggregation and consolidation of data on the occurrence and distribution of many organic and inorganic contaminants and background constituents of drinking water an important process. These data can then be made available for systemization and visualization to regulators, municipalities, water utilities, public interest groups, health researchers, consulting engineers, and water treatment scientists.

Given a sufficient number of data points and a convenient computerized database/mapping platform, a wide variety of maps can be generated to use in research and decision-making processes. The validity of doing so, however, rests inseparably upon the basis of the sampling plan and protocols, as well as the precision and accuracy of the analytical methods used for the constituents of interest. The well-known problem of matching the proper scale of the source data to that employed in the maps for interpretation is a critical problem with drinking water sampling, where many unappreciated small-scale variations render many, if not most, attempts to make generalizations inaccurate or meaningless.

This paper introduces and describes many concepts related to what generates or controls the concentrations of metals and other constituents in drinking water, ways in which the sampling protocol affects apparent levels of constituents, and the magnitude of temporal and spatial variability present in both municipal and private water supplies. Illustrations from water quality studies show in practical terms how generalizations must be kept to a minimum and how the data input into a geographic information system (GIS) for interpretation and evaluation must be carefully analyzed and screened to determine the appropriateness for various well-intended purposes. The discussion and examples show how many apparently significant trends and assessments of exposures or occurrences turn out to be merely artifacts of critical (yet subtle) inconsistencies or errors in the planning and execution of the sample collection process, or inconsistencies caused by the fact that regulatory (and not research) requirements govern the origin of the data.

The concepts this paper covers are equally valid in many other disciplines using or contemplating the use of GIS for interpretation of all kinds of "field" data.

Why Maps Are Useful for Drinking Water Studies

Maps and GIS databases could have wide applicability to drinking water studies. For example, they could provide the basis for investigating the occurrence of regulatory contaminants or related constituents, either to estimate the costs of compliance with a regulation or to estimate human health effects. Mapping could be useful to utilities and consultants investigating process changes for a utility or determining the effectiveness of some existing treatment such as corrosion control or chlorination. Use of GIS could also assist in assessing the feasibility and impact of system expansion. Another promising application would be GIS assistance in developing and implementing wellhead protection plans. Many other areas of application may be possible now, or will be discovered in the future, as GIS technology and regulatory requirements continue to develop.

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Sampling Protocols for Data Usable in GIS

Several SDWA regulations have resulted or will result in the collection of geographically diverse drinking water quality data that may interest mappers. The Lead and Copper Rule, the Surface Water Treatment Rule, the proposed Information Collection Rule, and the Disinfection/Disinfection Byproduct Rule are but four examples. Many states have their own variations on federal drinking water regulations, so their data collection requirements may differ somewhat. Considerable data may also be collected for specific research studies of either academic or purely practical nature.

Chemical Factors in Constituent Behavior

For the purposes of this discussion, chemical constituents in drinking water may be classified as being generally reactive or nonreactive. Reactive constituents may change concentrations or chemical form for a variety of reasons, such as:

- A result of interaction with the background composition of the drinking water.
- By precipitation or dissolution reactions with pipe material used for the distribution system.
- By chemical reactions with disinfectants added at water treatment plants.
- By slow chemical reactions started at water treatment plants.

Nonreactive constituents may play an important role by providing a chemical background that indirectly influences the speed or extent of other chemical reactions and transformations. Table 1 gives a summary of many common constituents of drinking water and identifies whether they function essentially as reactive or nonreactive constituents.

Table 1. General Reactivity Trends for Common Drinking Water Constituents

Constituent	General Reactivity Tendency
pH	Highly reactive
Dissolved oxygen	Reactive
Calcium	Nonreactive (reactive when cementitious pipe linings are present)
Magnesium	Nonreactive
Total carbonate	Nonreactive
Total alkalinity	Reactive, particularly with pH changes
Chlorine residual	Reactive
Temperature	Either
Iron	Reactive
Copper	Reactive
Lead	Reactive
Zinc	Reactive
Silica	Nonreactive
Sulfate	Nonreactive
Orthophosphate	Reactive
Polyphosphate	Reactive
Total phosphate	Reactive
Nitrate	Nonreactive
Chloride	Nonreactive
Fluoride	Nonreactive
Trihalomethanes	Reactive
Haloacetic acids	Reactive

Reactive Constituents

Clearly, chemical species or compounds that can change in concentration or transform into other species or compounds during distribution make mapping on very large scales difficult to justify. Reactive constituents may also change concentration in the same place over time, such as water standing overnight in a home, school, or building, which is discussed in a later section. Some examples of reactions during water distribution follow:

- During lime softening processes at some central water treatment plants, a supersaturated state is used for the compound calcium carbonate to remove calcium (and sometimes magnesium) ions from the water. This condition is sometimes maintained into the distribution system as well to assist in maintaining chemical conditions useful for corrosion control of lead and copper. Thus, calcium levels, pH, and carbonate concentrations (and consequently, alkalinity) drop as water passes away from the plant (1, 2).
- The metals in pipe materials, such as iron, copper, zinc (in galvanized pipe), and lead, are oxidized by oxygen, free chlorine, chloramines, ozone, and other disinfectants, which renders them into a form that water can transport, unless other chemical conditions are such that a highly insoluble scale deposits on the pipe, immobilizing the metal (1, 3).
- Prolonged contact with chlorine disinfectant species converts a fraction of natural organic matter present in many distributed waters into regulated "disinfection byproduct" compounds, such as trihalomethanes, chloroform, and haloacetic acids (4, 5).
- Following the addition of chlorine or after increasing pH to enable some corrosion control for copper and lead, iron present in well waters in dissolved ferrous (Fe^{2+}) form oxidizes into Fe^{3+} form, which is much less soluble. Obnoxious "red water" results, as ferric oxyhydroxide precipitate forms and clouds the water.
- Polyphosphate chemicals added to "sequester" iron or manganese in well waters break down into simpler

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polyphosphate forms of shorter chain lengths, plus orthophosphate. The orthophosphate frequently becomes present at high enough concentration to aid in controlling lead or copper (1, 6-8).

- Water passes through newly installed cement mortar-lined pipes, or aggressive water passes through older asbestos-cement pipes. Because of the particular chemical nature of the water, calcium carbonate and calcium hydroxide in the cement dissolve, raising the pH and hardness of the water (1).
- Free chlorine is added to disinfect water and is such a strong oxidant that it is unstable in water at normal concentrations. Additionally, it reacts with miles of unlined cast iron pipe, accelerating the decomposition of hypochlorous acid or hypochlorite ion to chloride. Consequently, the overall redox potential of the water supply and the effectiveness of disinfection decrease.
- A concentration of 1 milligram per liter (as PO_4) phosphoric acid is added to a distributed water at pH 7.5 to control lead corrosion. The orthophosphate reacts with exposed iron in the distribution main, however, and the residual concentration of orthophosphate decreases throughout distribution passage to the point where the level is no longer adequate to create the lead orthophosphate passivating film needed (1, 6, 8, 9).

Unless a constituent is known to be nonreactive, maps may be falsely generated under the premise that the concentration of a constituent is essentially a constant over some geographic area. Following the changes in concentration or chemical form of reactive constituents would also seem to be a useful application of GIS technology. One major restriction applies to the viability of that approach, however. Presuming that the analytical techniques used can adequately quantify the concentration and concentration changes observed, the scale of the variability or concentration change relative to the scale of the mapping perspective becomes critical to accurate mapping. A later section of this paper considers this critical factor in more specific detail.

Nonreactive Constituents

Almost no inorganic constituents in natural or drinking water are purely chemically inert. Under some conditions, and at some concentrations, significant reactions can occur. Some constituents that are actually reactive may act as if they are nonreactive constituents, however, because they are present in high enough concentrations relative to the extent of chemical reactions taking place that no discernible change in their concentration results. An obvious example is the dissolved inorganic carbonate (DIC = $H_2CO_3^* + HCO_3^- + CO_3^{2-}$) concentration (1, 10). Complexation and formation of passivating basic carbonate solid films of lead and copper by carbonate and bicarbonate ion dominate the corrosion control chemistry of copper(II) and lead(II) (1, 11). The concentration of DIC in water on either a molar or weight basis, however, is normally a factor of 500 to 10,000 higher than the lead or copper concentrations. Hence, changes in the DIC content from these reactions normally are analytically undetectable.

Another example is fluoride ion, which is often used as a distribution system water flow "tracer" because of its relative inertness. Actually, fluoride ion can form strong complexes with aluminum left in water following coagulation treatment with alum. The solubility of fluoride-containing solids with other major drinking water components (such as calcium and sodium) is very high, however, and fluoride reacts only weakly with metallic plumbing materials in the distribution system. Therefore, total fluoride concentrations tend to remain constant.

Relatively accurate maps of the occurrence and distribution of nonreactive constituents can be made, but their usefulness depends on the scale of the mapping relative to their occurrence and the particular question under investigation. All of this supports the need to ensure that the question asked can be answered correctly at the map scale.

Scale of Drinking Water Constituent Sources

More than 59,000 public water suppliers exist in the United States (12). Of these, approximately 660 are considered large water systems, which serve over 50,000 in population. These municipal systems use source water supplies that can be ground-water wells, "surface" waters (i.e., rivers, reservoirs, lakes), or a combination of both. Some water suppliers perform minimal water treatment of their own and purchase water from another water system or systems to satisfy their needs.

Surface Water Sources

Many water utilities use a single water treatment plant to treat surface waters, which could satisfy the entire water demand of the community all year. In many cases, however, utilities combine several surface water sources and use a different treatment plant to treat each water source. The water plants usually discharge into the distribution system at different points, and system hydraulics dictate the areas of the system in which waters mix. This is important because the water quality characteristics, which often differ among treatment plants, influence the corrosivity of the waters to various plumbing materials in the distribution system. Different water constituents also may affect the disinfection effectiveness of the treatment and the formation of unwanted disinfection byproducts.

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For surface water systems, the chemical composition of the water depends on the upstream or watershed geochemistry, the seasonal nature of the water body used as the source, and the characteristics that the treatment imparts, such as coagulation with ferric sulfate or alum (aluminum sulfate), lime softening, filtration, pH adjustment, corrosion control treatment, chlorination, etc.

The scale of the source water chemical data, therefore, is large, driven by the geology, soil nature, land use, and climate. The chemical nature of the treated water, however, may differ significantly from that of its source.

Ground-Water Supplies

Many water utilities use multiple ground-water wells. A water supply of medium to large size usually uses multiple wells, instead of or in addition to the surface water supplies. Wells number from only two or three to more than 100 for very large water systems. Wells normally operate in different patterns, and only rarely do all wells operate at the same time. The yield of the wells and their water quality dictates the combination and number of wells used at a particular moment. The wells may or may not be from the same aquifer, and even if they are, local inhomogeneities frequently exist in water composition (especially with iron and manganese) that limit the usefulness of certain wells without substantial treatment.

Historically, utilities have treated some (but not necessarily all) wells with a chemical such as a polyphosphate or sodium silicate to sequester the iron and manganese from wells. Some utilities install physical removal processes such as ion-exchange softeners, reverse-osmosis plants, aeration systems for iron removal, air stripping towers for volatile organic compound or radon removal, or "greensand" filters for the removal of iron and manganese. These facilities sometimes exist at only certain well sites or at some point where water from multiple wells is combined.

The scale of chemical controls on ground-water supplies, therefore, becomes only hundreds of feet. Contaminants of raw waters, such as arsenic, nitrate, or chromium, are geologically and geochemically controlled. Therefore, their occurrence is geographically variable on even a small scale, and the variability exists vertically in the subsurface as well as horizontally. A municipality may use wells of different depths into different aquifers, or even approximately the same depth spread out over hundreds of feet to many miles in the same aquifer or a variety of geologic units.

The variability of individual ground-water wells over time (such as seasonally) is usually less apparent than with surface water sources, but the fact that many wells are frequently used in different combinations and for different lengths of time (hours to days, usually) makes characterizing "influent" water quality complicated. The same observation applies to water systems that allow different amounts of water to bypass treatment processes (e.g., ion-exchange, reverse osmosis) depending on the levels of targeted undesirable contaminants (e.g., nitrate, sulfate, arsenic).

These characteristics of the nature of chemical composition, use, and treatment of ground-water supplies clearly show that generalizations over areas such as states or geographic regions (e.g., New England, Upper Midwest) are at least very gross and uncertain and at worst, entirely misleading when decisions are to be made about risk and health assessments, or estimates of the necessity for certain treatments or economic impacts of different potential drinking water regulations.

Combination Systems

Some municipalities combine the use of surface water supplies and ground-water wells. Therefore, general water chemical characteristics vary throughout the system in a regular manner in response to the location and use of different sources, as well as relative amounts of water that the different sources produce and deliver.

Distribution System Mains

The next lower level of scale is the distribution system network of pipes and storage. Common materials used for distribution system piping include cast iron, ductile iron, cement mortar-lined iron, iron with organic coatings, asbestos-cement (A-C), and various forms of plastic. Pipe diameters range from about 4 inches to many feet, depending on size of the water utility and community, size of the neighborhood fed by the line, and distance of travel for the water. Here, because of the large volume of water involved relative to the pipe diameter, the major chemical interactions involve such constituents as hardness (calcium and magnesium) ions, pH, iron, bicarbonate and carbonate ions, and chlorine residual species, and possibly microbiological parameters such as total plate counts, heterotrophic plate counts, and assimilable organic carbon. Disinfection byproducts (DBPs) may change in concentration and type because of the time involved in the water traveling through the piping from the treatment plant. Trace metal contamination, such as lead and copper, is usually negligible from this source, unless it is present when distributed from the wells or water treatment plants.

Depending on prevalent economics and construction practices during periods of water system growth, the materials will not be either randomly or uniformly distributed geographically within system boundaries. Water flow often varies greatly within the distribution system, and water lines sometimes terminate in dead-end areas with minimal flow rates. Water quality often differs substantially in these dead ends from that in the fully flowing parts of the distribution system.

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Household Service Lines

Service lines represent the connection between the house or building and the distribution main. Sometimes, the service lines are joined to the mains by a flexible, approximately 2- to 3-foot long pipe called a "goose-neck" or "pig-tail." Historically, this connector was often made of lead. Recently, copper has been the most widely used material, with plastic gaining in acceptance. Service lines for homes are usually 0.75 to 1 inch in diameter, with service lines for many commercial buildings or multifamily dwellings ranging in size from 1.5 to 3 inches in diameter. Service line for homes and buildings have usually been made of lead, brass, copper, galvanized steel, or plastic. The material used depends on the age of the water connection and the construction practices of the area involved. A recent report estimated that approximately 6.4 million lead connections ("goose-necks") still exist in the United States, and about 3.3 million lead service lines still exist (13). In many communities, old lead service lines remain a major source of lead in drinking water.

Like distribution system materials, service line materials may vary greatly within a distribution system by space and time. For instance, in large eastern cities, very old neighborhoods may have many (or even mostly) lead service lines. New neighborhoods likely have copper or plastic service lines. Galvanized steel or copper pipes may have been installed between the era when lead was used and modern times. With the exception of Chicago, where lead service lines were occasionally installed into the 1980s, the use of lead for service lines generally stopped in the late 1940s or early 1950s. An example of nonuniform distribution of service line materials is shown by Figure 1, a map indicating Cincinnati sampling sites for Lead and Copper Rule (14-17) monitoring. Erratic clustering of different service line materials is evident.

Rehabilitation of old houses or replacement of failed piping results in a mixture of new and old material in areas where houses are predominantly old. Following completion of the construction, maps of service line material would show many clusters representing prevalent plumbing codes and economics.

Interior Plumbing

Interior plumbing of buildings and houses reflects even more variability than service lines. This is the dominant contributor to lead and copper levels at most sites covered under the Lead and Copper Rule (14-17). Interior plumbing consists of piping, plus a large number of valves, connectors, fixtures, and perhaps soldered joints and a water meter. Any or all of these components are replaced at varying intervals as a result of failures or remodeling. Therefore, even generalizations within a small neighborhood are risky, unless the neighborhood is very new and uniformly constructed. When attempting to survey the composition of plumbing materials that might be the source of drinking water contamination, merely asking for the age of the house or building is insufficient. Questions must be asked to obtain the necessary precise information on the age and type of plumbing materials and components in the building.

Typical interior plumbing materials include lead, galvanized steel, copper, and different plastics for pipes. Some brass and black steel have been used for short times in some areas. Faucets are almost always made with either brass or plastic internal parts, which differ in composition from the exteriors, which are usually plated with chrome or other metal. Interior faucet volumes typically range from about 30 milliliters to 120 milliliters, depending upon design. Valves and meters are also frequently made of brass or bronze, which are copper/zinc alloys usually containing 2 percent to 6 percent lead. Until recently, solders used to join copper drinking water pipe sections were usually a tin and lead combination, containing 40 percent to 60 percent lead. Occasionally, connector lines to fixtures include copper, stainless steel, aluminum, or flexible plastic sections.

Private Water Systems

The many possible designs of domestic water systems originating from wells or cisterns are too numerous to illustrate. Figure 2 gives an example of one such system layout. Private systems share many features with domestic systems supplied by water utilities, however. Interior plumbing shares most of the same configurations and materials. For private water systems, additional plumbing that could cause contamination or water chemistry changes includes well casing material, submersible pump casing and fittings, pressure tank feed and control plumbing, and nonsubmersible pump interior materials. Therefore, problems with determining the frequency and distribution of levels of potential contaminants include those present for domestic situations in general, plus those complications arising from cycling of the pumps, pressure tank system, or both.

Water Samples Representing Distances

One of the most important fundamentals of understanding drinking water sampling is that volumes of water (e.g., 1-liter samples, 250-milliliter samples) represent the linear distance of plumbing material in contact with the water sampled. Because of water mixing and flow during use or sampling, they are also integrated samples of that volume. This understanding is at the heart of designing accurate water sampling programs and making viable interpretations of existing monitoring data that may be contained in (or mappable by) a GIS.

Table 2 summarizes some interesting and important relationships between pipes of different inside diameters

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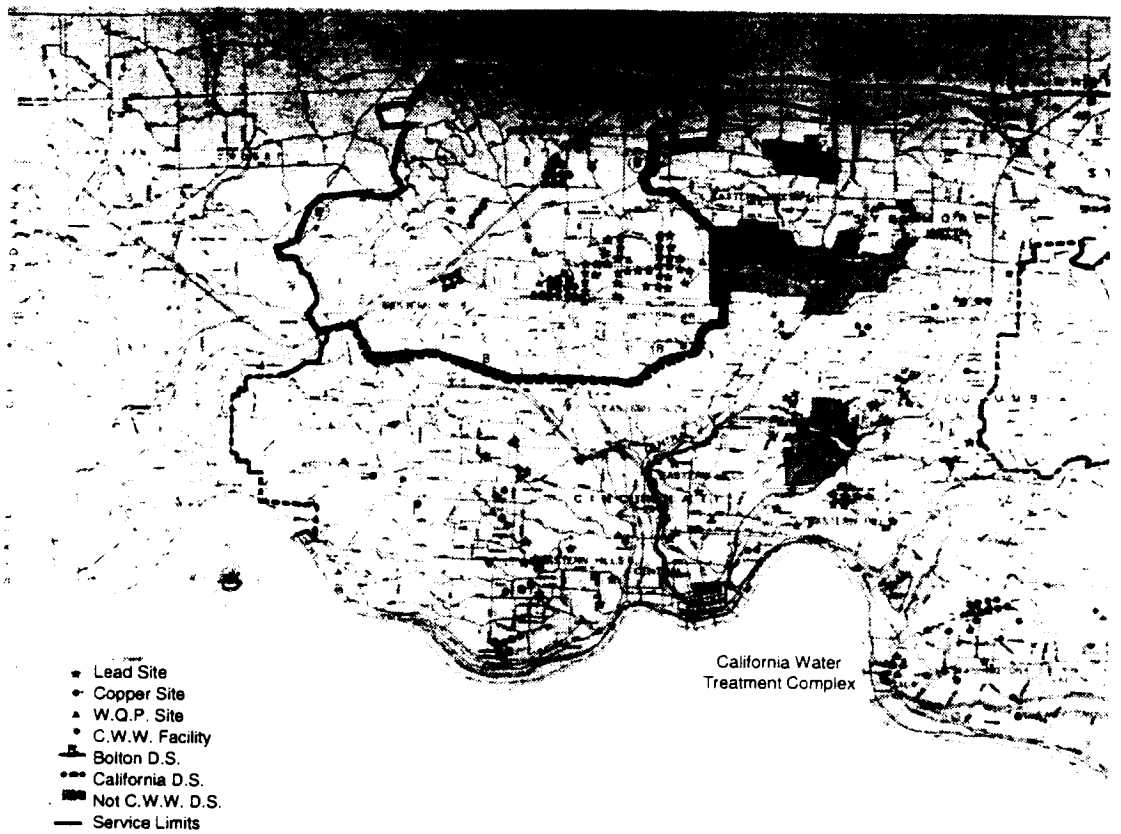

Figure 1. Cincinnati Water Works Lead and Copper Rule compliance monitoring, July to December 1992.

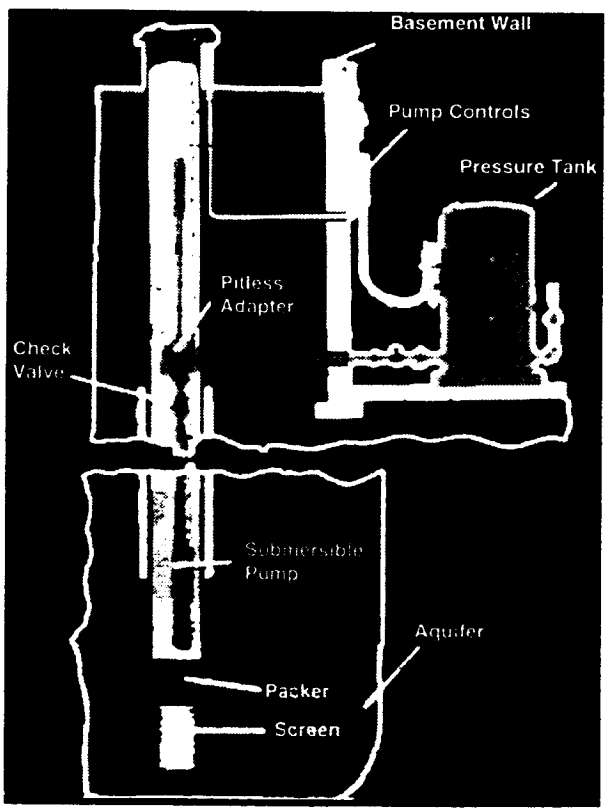

Figure 2. Distribution system.

(IDs) and the volumes of water they contain per unit of length (7). Much domestic interior plumbing has an ID of approximately 0.5 inches, depending upon the material.

Figure 3 shows schematically what parts of a plumbing system would likely be represented by samples of different volumes taken after water was allowed to stand in the pipe for many hours. Faucets, bubblers, and other terminating fixtures vary widely in volume. Kitchen-type fixtures usually contain from 60 to 120 milliliters of water. Bathroom-type fixtures may contain only about 30 to 60 milliliters of water. Bubblers, such as those frequently found on school or office drinking fountains, are smaller still. As can be seen schematically in Figure 3a, a small volume such as 125 milliliters captures the faucet and a short distance of pipe immediately leading to it. In many plumbing systems, this volume catches water in contact with numerous soldered joints. On the other hand, if a single 1-liter first-draw sample is taken, the water in the bottle represents a much longer distance back into the plumbing system. In a situation where the source of lead in drinking water is a new brass faucet, or soldered joints of lead-tin solder, this larger volume usually gives a lower lead concentration than the smaller volume because more water in the sample is not in intimate contact with materials containing lead.

Other sampling schemes logically follow. For instance, if examining copper pipe corrosion, discarding the first

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Table 2. Interrelationships Among Pipe Length, ID, and Internal Volume for Selected Common Plumbing Materials and Pipe Sizes

Material	Identification/ Type	True ID	True OD	Length for 1,000 Milliliters (Feet)	Milliliters per Foot
Copper tubing	0.5-inch, type L, annealed	0.545	0.625	22	46
Copper tubing	0.5-inch, type L, drawn	0.545	0.625	22	46
Copper pipe	0.5-inch, schedule 40	0.622	0.840	17	60
Galvanized steel pipe	0.5-inch, schedule 40	0.616	0.840	17	59
Lead pipe or tube	0.5-inch ID, 0.25-inch wall	0.50	1.00	26	39
Lead pipe or tube	0.75-inch ID, 0.25-inch wall	0.75	1.25	11.5	87
PVC or CPVC pipe	0.5-inch, schedule 80	0.546	0.840	22	46

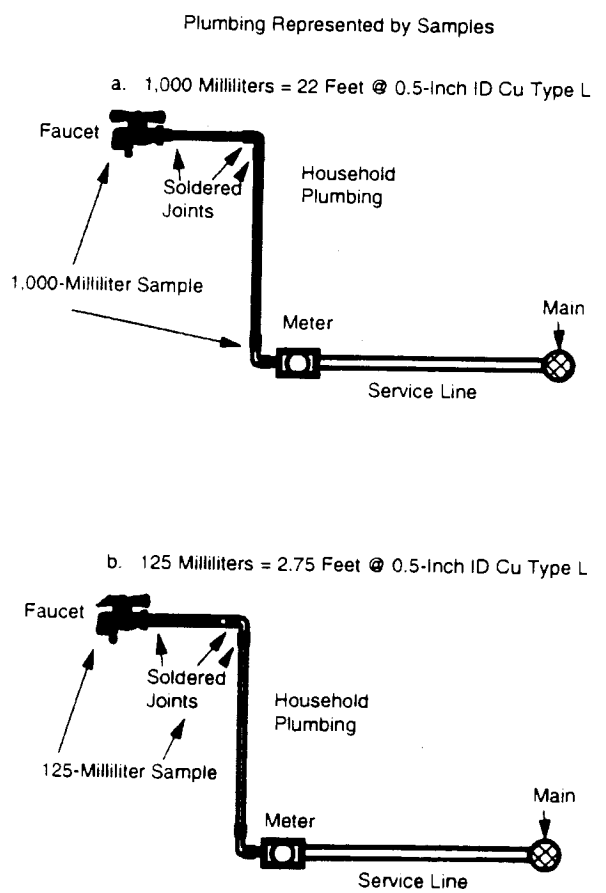

Figure 3. Schematic diagram of plumbing materials represented by sample volumes of a) 1 liter and b) 125 milliliters.

125 or 250 milliliters of water is likely to give more accurate information because it minimizes the effects of the faucet material as well as piping that connects the faucet to the interior line. Often, this connecting piping is not copper. If examining the corrosivity of the water to lead service lines, wasting a volume of water corresponding to the distance from the outlet to the service line better estimates the effect, although not without uncertainty (18). Many other sampling schemes are possible and useful, but users must be aware that the sampling protocol may have as much or more influence on the observed metal concentration than water quality or other variables. Hence, incorporation of monitoring data into a GIS database must be done only when the source represents equivalent samples.

Because of turbulent mixing during flow, local high concentrations of lead (or other contaminant) may become broadened and diluted by the time the water to be sampled reaches the sample collection bottle (18). In many cases, therefore, numerous small-volume sequential samples can be taken and used to profile a plumbing system to locate brass valves, connectors, soldered joints, etc. Figure 4 illustrates sequential sampling results for one room of a building. Peaks in the distribution of samples physically correspond to the location of a chrome-plated brass faucet and to a later concentration of fresh Sn:Pb soldered joints. Unfortunately, even small-volume sequential tap water samples must pass over other potentially contaminating or altering surfaces on the way through the sampling tap.

Kuch and Wagner have shown how water can dissolve large amounts of lead simply by traveling through long distances in lead pipes with small IDs (9, 19). Although this study specifically examined lead, the principle applies to other metallic piping materials. This phenomenon is inseparable from the aspect of time, which is the next subject.

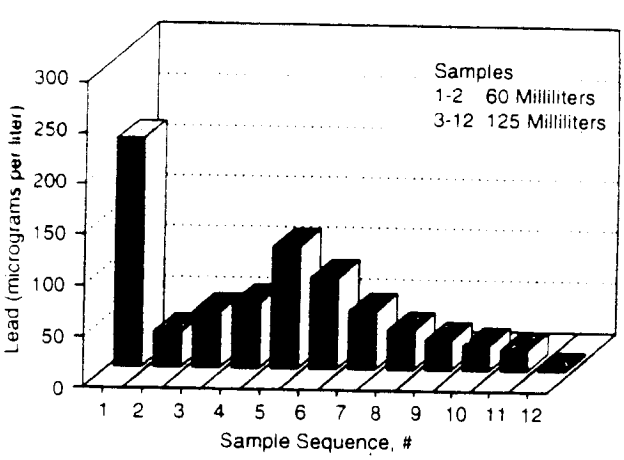

Figure 4. Sequential sampling results from a room on the ground floor of a building.

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Effect of Time

While some chemical reactions are instantaneous, many dissolution and precipitation reaction steps that are important in controlling metal levels in water take many hours to many days to reach equilibrium. In fact, passivation films and scales on pipes that inhibit corrosion and reduce leaching of trace metals may take months to decades to develop substantially. Some other chemical transformations, such as creation of trihalomethanes from chlorination of natural organic matter during disinfection, or processes such as inactivation of pathogens, may occur over hours (20, 21).

Many steps in an overall chemical reaction process could be rate-limiting. Figure 5 shows how lead levels increase in 0.5-inch ID pipe given two different assumptions. The top curve shows how lead increases and levels off after about 8 to 12 hours (9, 19). This curve is closely applicable to any metal, as long as the limiting factor on the rate of metal migration into the water is the radial diffusion of the soluble metal species away from the pipe surface. The second curve shows, schematically, the effect of a diffusion barrier film (e.g., calcium carbonate, adsorbed iron hydroxide mixed with organic matter, aluminosilicate mineral deposits) or inhibition of metal oxidation rates on lead migration into water after different amounts of time.

In some water systems, significant chemical changes can occur while the water is standing that can drastically affect the oxidation or solubility of the plumbing material. For example, dissolved oxygen and free chlorine react quickly in new copper pipe or brass. If the water stands

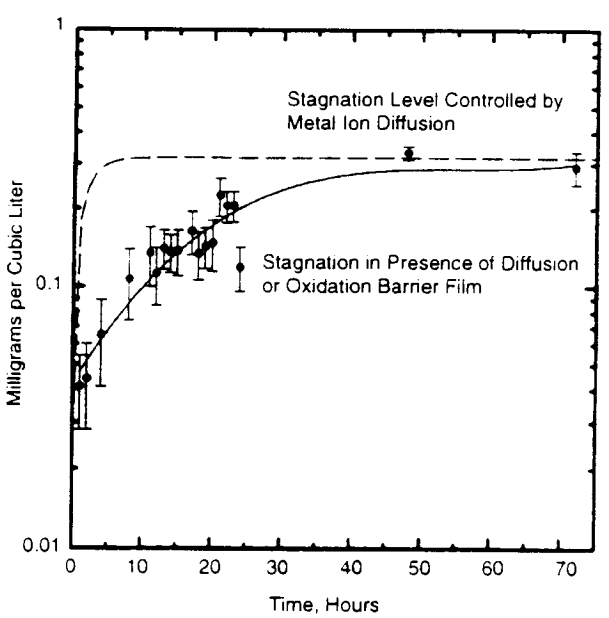

Figure 5. Comparison of lead concentrations that would be observed after water stands different amounts of time given different controlling chemistry factors.

sufficiently long, their concentrations may become negligible, which would significantly alter the redox conditions governing metal solubility. In the absence of oxygen or chlorine species, the dominant form of copper in water and on plumbing material then becomes copper(I) instead of copper(II), resulting in different solubility characteristics after consumption of the oxidant than at initiation of the standing period (11).

Seemingly identical water samples collected from the same taps in houses, schools, or other buildings yield different metal concentrations, depending on the time the water was in contact with the faucet, solder, or piping material. Similarly, samples taken for disinfection byproducts after different chlorine contact times may produce different concentrations and different speciation (e.g., trihalomethanes, haloacetic acids). This factor causes considerable confusion in many investigations of contamination of school or building drinking water taps and water coolers and complicates estimating human exposure for health-effects studies.

Interconnectedness of Distance and Time

In innumerable situations, the effects of distance and time are impossible to separate. Some generalizations and examples follow.

Dead ends and slow rate areas produce long residence times for the distributed water. This results in long contact times with pipe materials, so reactive constituents can change considerably in concentration. The process is totally interactive, in that concentration changes of reactive constituents are in response to contact with the pipe materials, and in turn, the materials respond to the water composition. Water may take hours to days to reach a particular home or building and may traverse many miles of distribution system piping of the same or differing composition. Water thoroughly run through a household faucet for 5 or 10 minutes to purge the lines is "fresh" from the resident's perspective, but may be "old" from the distribution system perspective.

The profile of the water line shown in Figure 4 was made on the basis of filling small volume (60 or 125 milliliters) sample bottles, one after another, without wasting any water. If the objective were to capture only the highest risk of lead contamination from a lead service line after some hours of stagnation, then the sampling process would be different. Instead of collecting all water between the tap and the service line, the water can be run until either a target volume is wasted (representing the linear plumbing distance to the service line) or can be run at a given rate until, after the appropriate length of time passes, the sample bottle can "intercept" the slug of water residing in the service line. Beware that differences may exist in the peak concentration of the contaminant and the "width" of the slug of elevated

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contaminant level, depending upon the rate of water flow before and during sampling (18, 22, 23).

Other Sources of Variability in Water Samples

Variability in water samples can stem from many sources aside from those discussed in this paper (18). The nature of the errors and their likely magnitude may vary with each episode of sampling and analysis and is far beyond the present scope of discussion. A brief listing to consider, however, when drawing conclusions from "field" data includes:

- Analytical imprecision or bias.
- Flow rate of the water during sampling.
- Temperature.
- Particulate erosion from plumbing materials during sampling.
- Effect of container material.
- Effect of air contact or other handling effects during sample collection and shipment.

A Case Study of Easy Misinterpretation

Interpretations of water quality problems based on aggregate monitoring data can be very misleading unless analysis is performed at the appropriate scale. The situation of one utility described below provides a good example of how using a GIS approach could have helped solve the problem but also highlights how carefully data would need to be matched and consolidated only at the proper scale if GIS were to be employed for evaluating some kinds of water quality problems.

The utility at Hopkinton, Massachusetts, found very high lead and copper levels exceeding the regulatory action levels under the Lead and Copper Rule (14-17). The 90th percentile copper levels even exceeded 6 milligrams per liter, compared with an action level requirement of only 1.3 milligrams per liter. Some sites with lead service lines are present in the system. Figure 6 shows a schematic representation of the distribution system for this utility. Five wells feed the system, and four (1, 2, 4, and 5) are used regularly.

The background hardness, alkalinity, and carbonate concentrations are fairly similar for all wells. The pH of the ground water from the wells is usually slightly above 6. Chlorine solution is dosed for disinfection. High iron levels are present in wells 1, 2, and 3, and high manganese is also present in well 3. Generally, high dosages of a polyphosphate chemical were added to wells 1 and 2 to respond to consumer complaints about the "red water" that results from iron oxidation and precipitation. Water from different wells mixes in the distribution system, but the water tends to partition into two zones as

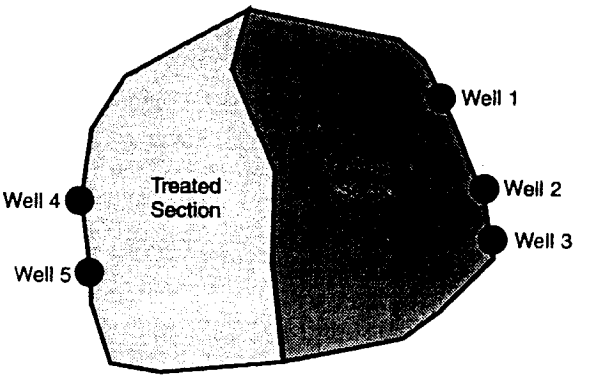

Figure 6. Schematic representation of the utility's distribution system.

Figure 6 indicates. The lead and copper levels tended to be distinctly lower in the section where the polyphosphate was dosed, marked as the "treated" part of the system. The ongoing research study has employed approximately 22 monitoring sites.

From the information presented thus far, the system clearly cannot be characterized by a discrete value for lead or copper contamination, as well as the chemical background of water throughout the system. Hence, putting data at the "whole system" scale into a statewide or countrywide data system would be tempting, but it could be very misleading in solving the treatment problem. Having accurate spatially distributed data for background water qualities, monitoring site characteristics, and metal levels at the subsystem scale, such as that which could be integrated into GIS, would have been extremely convenient, however. Yet, even more information at a smaller scale is necessary to understand and solve the whole treatment problem.

The utility initially observed that because the lowest lead and copper levels coincided with the area of the system fed by the polyphosphate chemical, that chemical likely caused the corrosion inhibition. Median lead levels, for example, were between about 200 and 300 milligrams per liter in the "untreated" section, compared with about 10 to 15 milligrams per liter in the "treated" section. Median copper levels were approximately 4 to 5 milligrams per liter in the untreated section, but only about 0.3 to 0.5 milligrams per liter in the treated section. The utility and the researchers wondered whether the polyphosphate chemical should also be added to the other wells. This is a matter of significant concern because some studies indicate polyphosphate chemicals can enhance lead corrosion (1, 6) and the subject has rarely been studied under statistically valid controlled conditions.

Additional site-by-site investigation, however, first revealed that the sites with lead service lines all lay in the untreated area of the distribution system. Because the

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research sampling program included two successive 1-liter samples, the additional contamination from the service lines was confirmed by higher lead levels in the second 1-liter sample than in the first in many cases. Therefore, physical reasons, in addition to chemical ones, explained the discrepancy in the lead levels. Further, when considering only the treated system sites, the lead levels were still high enough to be of concern.

Focusing on the copper sites resulted in the collection of more important and interesting small-scale information. Figure 7 shows the difference between average copper levels in the two sections of the system. Almost all sites in both parts of the system had copper interior plumbing with 50:50 or 60:40 Sn:Pb soldered joints and faucets with brass-containing internal materials. Though the chemical added for iron control was ostensibly a polyphosphate chemical, it also contained an initially present fraction of orthophosphate and also tended to partially break down to orthophosphate in the presence of iron and calcium (as most polyphosphates do). Figure 8 shows the orthophosphate concentrations in the two different parts of the system. While the levels of orthophosphate present in the treated section would be far too low to significantly inhibit lead leaching at the background pH (1, 6, 7, 9, 22), the orthophosphate plausibly may significantly inhibit copper dissolution, in concordance with recent research projections (11).

Having determined through detailed small-scale sampling and analysis that the chemistry affecting metal levels in the system is generally consistent with modern knowledge, a new treatment plan is being implemented to control copper and lead levels through pH adjustment in conjunction with iron control through a compatible sodium silicate/oxidation treatment. Incorporation of system and monitoring site physical characteristic data, plus monitoring results, into GIS could have saved considerable investigatory effort. The importance of this case history, however, is that the data must be of the appropriate scale and highly documented to be useful in problem-solving. Failure to use data meeting these requirements, as well as overgeneralization to a large mapping scale, can lead to ineffective if not damaging water treatment choices that could adversely affect public health.

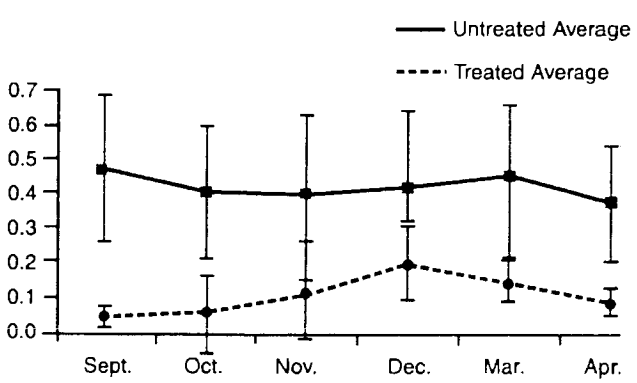

Figure 7. Average copper levels in treated and untreated sections of the system.

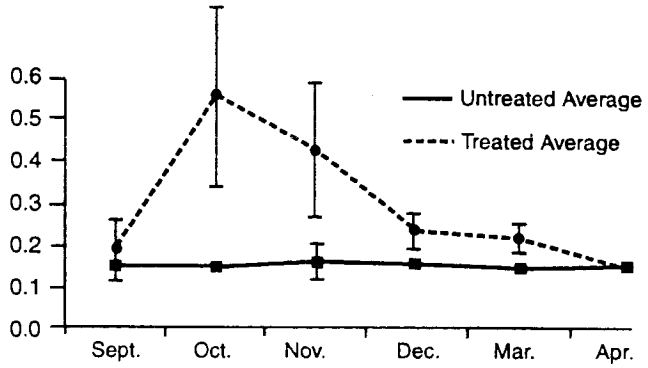

Figure 8. Average orthophosphate levels in treated and untreated sections of the system.

Conclusions

The examples and discussion above lead to several general conclusions about the use of GIS with drinking water monitoring data:

- Temporal and spatial variability stems from many causes, down to a very small scale.
- Sampling protocols must be keyed to the precise questions under investigation.
- Regulatory sampling, whose results are generally readily available, is usually inappropriate to assess human exposure to trace metals or other parameters of interest (such as DBPs).
- Generalizations on a large scale are often impossible because of the geology and water chemistry variations.

Additionally, some considerations apply to the types of mapping that could be employed by GIS. For example, a mapping technique such as contouring may be especially inappropriate for use with drinking water data. Major problems could result from:

- Discrete, small-scale (such as within an individual house) variability in distributions of certain contaminants, such as lead and copper.
- Physical constraints of the distribution system network.
- The small number of monitoring sites in relation to the size of the distribution network.
- Different chemical or hydraulic zones in the distribution system.

Employing GIS could be very useful in solving a variety of drinking water problems. Users must be extremely conscious of the nature of the source information, however, to avoid abusive extrapolations and generalizations.

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Acknowledgments

Some of the examples cited were investigated as part of a U.S. Environmental Protection Agency cooperative agreement (Darren A. Lytle, Project Officer) with the New England Water Works Association. Jack DeMarco of the Cincinnati Water Works kindly provided Figure 1 on distribution of sampling sites for lead and copper monitoring.

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Ground-Water Applications

Using GIS/GPS in the Design and Operation of Minnesota's Ground Water Monitoring and Assessment Program

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Abstract

Minnesota's Ground Water Monitoring and Assessment Program (GWMAP) is administered by the Minnesota Pollution Control Agency (MPCA) to evaluate baseline ground-water quality conditions regionally and statewide. The program uses a systematic sampling design to maintain uniform geographic distribution of randomly selected monitoring stations (wells) for ground-water sampling and data analysis. In 1993, geographic information system (GIS) and global positioning system (GPS) technologies were integrated into GWMAP, automating the selection of wells and the field determination of well locations.

GWMAP consists of three components: the statewide baseline network, regional monitoring cooperatives, and a trends analysis component. In the statewide baseline network, Minnesota is divided into over 700 121-square-mile grid cells, each with a centralized, 9-square-mile sampling region. Within each target area, single-aquifer, cased and grouted wells are sampled for about 125 metals, organic compounds, and major cations and anions. We are currently finishing the second year of a 5-year program to establish the statewide grid. When complete, the statewide baseline component will consist of about 1,600 wells representing Minnesota's 14 major aquifers.

In 1993, approximately 4,000 well construction records were selected for geologic and hydrologic review, using a GIS overlay, from a database of 200,000 water well records maintained in the state's County Well Index (CWI). Using GPS, 364 wells were sampled and field located. The semiautomatic well selection process uses existing electronic coverage of public land survey (PLS) data maintained in CWI in conjunction with the digitized systematic sampling grid. GIS has greatly reduced the time needed for selecting sampling stations. With the combination of GIS and GPS, program costs have decreased, allowing more resources to be applied toward sampling, while efficiency and quality of data have improved.

Introduction

Quantitative assessment of ground-water quality conditions requires a highly organized data collection program that includes statistical evaluation of monitoring results (1, 2). States have difficulty providing the staff and financial resources necessary to generate statewide quantitative ground-water information. With the use of geographic information system (GIS) and global positioning system (GPS) technologies, however, states have the potential to improve the quality of environmental monitoring programs and to reduce the amount of staff time needed to collect and evaluate data, thus decreasing costs. The degree to which states realize these potential benefits depends largely on how effectively the technology can be incorporated into the design of the monitoring program. This paper describes how GIS and GPS technologies are being integrated into the design and operation of Minnesota's Ground Water Monitoring and Assessment Program (GWMAP) to improve overall effectiveness.

The Minnesota Pollution Control Agency (MPCA) has sampled and analyzed ambient ground-water quality in the state's 14 principal aquifers since 1978. In 1990, the MPCA began a redesign of its ground-water monitoring program to better assess water quality conditions statewide (3). Three program components resulted from the redesign: a statewide baseline network for complete geographic coverage, a trends analysis component for intensive studies of how ground-water quality in specific areas changes with time, and a regional monitoring cooperative link to governmental units such as counties

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to meet specific local ground-water assessment needs. This paper describes the design and operation of the statewide baseline network.

The design of the statewide network is geographically and statistically based to automate well selection and data interpretation. In 1993, the MPCA began integrating GIS and GPS technologies into this part of the program. The implementation of GIS and GPS surpassed our expectations by reducing staff time required to select wells and evaluate analytical results (see Table 1). In addition, through the elimination of previously uncontrollable variables, the use of GIS and GPS has increased the accuracy of GWMAP data.

Monitoring Program Description

Since 1992, GWMAP has selected 150 to 250 existing water supplies yearly for ground-water sampling and analysis of about 125 parameters, including major cations and anions, metals, and volatile organic compounds. Well selection is a fundamental element of GWMAP that, if efficiently performed, supports the program objectives by upholding the quality of the monitoring data and minimizing the operating costs.

A key to the interpretation of monitoring data is the technique used to select wells for sampling (2, 4, 5). Minnesota has over 200,000 active water wells with approximately 10,000 new installations annually. For each well selected for GWMAP monitoring, a hydrologist must individually review many well construction records. An automated prescreening mechanism to facilitate well selection can result in considerable time (and therefore cost) savings. GWMAP chose GIS as the best tool for this task. GIS enables the program to combine a systematic sampling technique with hydrogeologic criteria to ensure an efficient and consistent selection process. As Table 1 shows, GIS allowed us to more than triple our geographic coverage and wells initially selected, while dramatically reducing the records that must be individually reviewed. We realized a time savings of 2 months compared with the time required before GIS implementation.

In general, systematic sampling techniques use a randomly generated uniform grid to determine sampling locations in space and/or time (5). Systematic sampling was initially implemented in GWMAP in 1991 using a manually generated spatial grid defined by the public land survey (PLS) (3). Although the PLS is not 100 percent geographically uniform, it was selected for the grid to expedite well selection from existing digital databases in which wells are organized by PLS location.

Systematic Sample Site Selection Using GIS

Systematic sample site selection is a three-step process. First, a database search of Minnesota's County Well Index (CWI) (6), containing nearly 200,000 driller's records, is conducted to include all available water wells in the region of interest. Second, the candidate pool is reduced to those wells located within regularly spaced grid cells. Third, further wells are eliminated from the candidate pool by applying geologic and well construction criteria mandated in the GWMAP design (7).

Generating the Sampling Grid

The statewide sampling grid was generated from a randomly selected origin (8). This grid consists of approximately 700 square cells, 11 miles on a side (see Figure 1). The centroid of each cell is consecutively numbered and was extracted to produce the origin of the sampling zone.

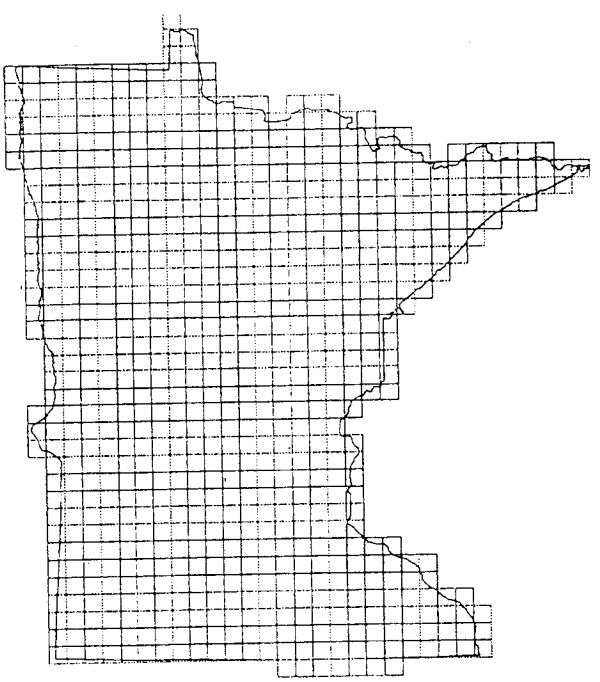

Figure 1. Statewide baseline network sampling grid.

Table 1. Well Selection in 1992 and 1993

Year	Area Covered	PLS Sections Selected	Well Logs Selected	Well Logs Reviewed	Wells Sampled	Time Spent
1992	9 counties	500	3,000	3,000	158	6 months
1993	26 counties	1,659	11,000	834	206	4 months

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Establishing the Sampling Zone

Each sampling zone consists of a 3- by 3-mile box from which potential sampling sites are selected. It is generated by computing the coordinates of the four corners of the box using the grid cell's centroid as the origin. To link the sampling zone and grid cell, both are identified with the same numerical code.

These sampling "target" zones, a series of regularly spaced, 9-square-mile boxes, are then made into a GIS coverage and overlaid on top of the PLS coverage to extract those sections that are associated with each of the sampling zones. Ideally, each sampling zone should cover exactly nine PLS sections (3). Due to irregularities in the PLS system, however, portions of 16 to 20 sections usually fall within the sampling zone of each cell (see Figure 2).

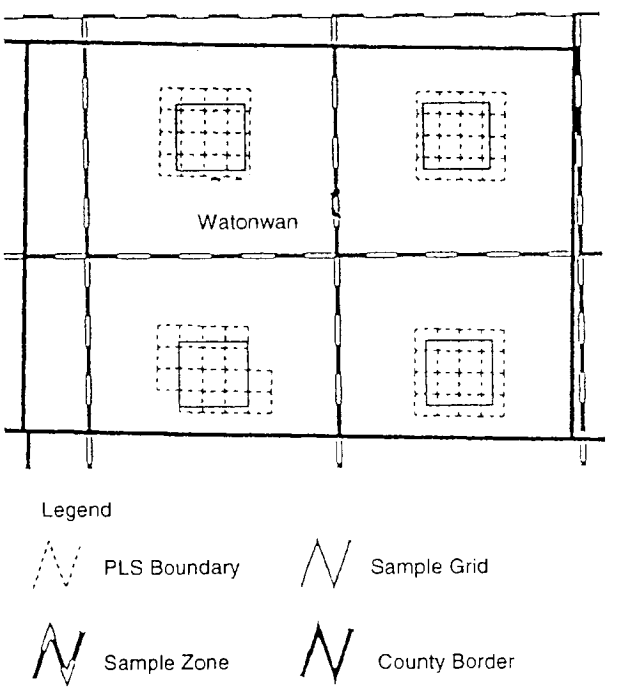

Figure 2. PLS and the sampling grid, Watonwan County.

Selection of PLS Sections

The PLS coverage was derived from the Minnesota Land Management Information Center (LMIC) "GISMO" file. It was originally created in 1979 by digitizing every section corner in Minnesota from the U.S. Geological Survey (USGS) 7.5-minute quadrangle map series.

The PLS section information is necessary in the well selection process because the original well construction logs, maintained by the Minnesota Geological Survey (MGS), are organized by PLS. Although most of the well selection process can be automated, manual file searches for well records are still necessary and require the PLS information.

Well Selection

After identifying the PLS sections within the sampling grid, the statewide well database is imported as a point coverage and overlaid with the selected PLS section coverage. Thus, all wells that fall within the 16 to 20 sections are selected as potential candidates. The accuracy of the well locations in CWI varies; most of the point locations are approximated to four quarters (2.5 acres). The CWI does not contain all well construction information, however, requiring that copies of driller's logs be made for GWMAP files.

The final well selection is done after applying the 9-square-mile sampling zone over the potential pool of candidates. For wells that fall within the zone, the well construction records are pulled from MGS files, copied, and submitted for hydrologist review. Depending on the target cell location, the number of candidate wells requiring review may range from a few to more than 100. For newly installed water wells whose records have not yet been digitized by LMIC, the PLS locations of the wells are manually plotted onto a map to confirm whether they fall into a sampling grid cell. Typically, from 5 percent to as many as 20 percent of selected wells that meet the location criteria are sampled. This accounts for the hydrogeologic and well construction criteria and the cooperation of well owners participating in the program. Currently, interest in ground-water protection programs runs high in rural Minnesota, with an acceptance rate of up to 80 percent.

The implementation of GIS in well selection helped GWMAP excel in two major areas. First, the development of the statewide GIS grid eliminated previously uncontrolled variables by removing the PLS spatial inconsistencies from the systematic grid. Second, the GIS reduced the manual workload with the automation of two important steps in the well selection process: the generation of PLS section information to facilitate the database search, and the identification of wells that meet the geographic location criteria. The success of GWMAP relies largely on the ability to use existing GIS coverages. In using coverages created by other entities, this program identified the need for a uniform standard for data conversion and transfer.

Application of Global Positioning Systems in Ground-Water Sampling

In 1991, the U.S. Environmental Protection Agency (EPA) established a policy that all new data collected after 1992 should meet an accuracy goal of 25 meters or better (9). The purpose of EPA's Locational Data Policy (LDP) is to establish principles for collecting and

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documenting consistently formatted locational data to facilitate cross-programmatic, multimedia analyses. Accurate geographic information is important to the spatial analysis of well sampling results. Any uncertainty in sample location can compromise hydrogeologic analysis (10). GPS is an easy, cost-effective solution.

Global Positioning System Field Application

Beginning in October 1992, GWMAP employed GPS in the field to assist in locating sample sites. Applying GPS in the field has proven to be quite easy. The program uses a multichannel C/A code receiver with internal data logging capability. Typically, the receiver is placed directly on top of the wellhead and logs 100 to 150 GPS readings into the receiver's internal memory in approximately 5 minutes.

The GPS is also used for navigation in the field to locate sampling sites. Because sampling sites are predetermined, their locations can be extracted from a topographic map. The approximate coordinates can then be loaded into a GPS receiver. In most cases, the receiver successfully led the field team within visual range of the sampling site.

Because of the inherent selective availability (SA) of the GPS, raw field data must go through a differential correction process to achieve the goal of 25-meter accuracy (9, 11).

Data Management and Processing

Once the GPS receiver is brought back from the field, data are downloaded to a personal computer (I486 processor at a speed of 50 MHz) and differentially corrected (11). The average or mean of the 100 or more readings collected onsite is calculated and reported as the site location.

The MPCA does not operate a GPS base station for the purpose of differential correction. The base station data are obtained through a computer network (Internet) from the Minnesota Department of Health (MDH) base station located in Minneapolis.

To facilitate future data integration and document data accuracy for secondary application, GWMAP proposed quality assurance codes for GPS data collected by the MPCA. The value of the accuracy proposed is a nominal value rather than an absolute number (see Table 2). Each of the seven processing methods is assigned a separate code.

In the field experience of GWMAP, a nominal accuracy of 2 to 5 meters has been consistently achieved after the postdifferential correction and averaging have been applied to the data. This technology is suitable for any program that is designed to conduct either large-area or intensive monitoring activities. It helps to cut costs by increasing efficiency and accuracy of the data. The data collected by GWMAP can be used not only in a regional study but could be used directly in a site-specific investigation as well.

Table 2. Proposed Nominal Accuracy Reference Table

Type of GPS Receiver Used	Processing Method Used To Correct Data	Nominal Accuracy (meters)
Navigational quality C/A code receiver	Postdifferential corrected	2-5
	Real-time differential corrected (RTCM)	2-5
	Autonomous mode (no correction)	15-100
Navigational quality with carrier aid receiver	Postdifferential corrected	< 1
	Real-time differential corrected (RTCM)	< 1
	Autonomous mode (no correction)	15-100
Survey quality receiver (dual or single frequency)	Postdifferential corrected	< 0.1

GWMAP also found that GPS can be used most efficiently by separating the two roles of field operator and data manager. The field operators receive only the brief instructions necessary to operate a GPS receiver before going into the field. The data manager handles the data processing details. The field operators can then concentrate their efforts on obtaining ground-water samples and conducting the hydrogeologic investigation.

Conclusions

GIS and GPS technologies made it possible for the MPCA to implement the statewide GWMAP project by optimizing the available funding and staff time. GIS minimized staff time spent on identifying sampling areas, manipulating the sampling grid, and selecting monitoring sites. In addition, GIS enabled GWMAP to integrate a variety of databases and maps of different scales.

Using GPS to locate sampling sites enabled GWMAP to efficiently obtain accurate geographic locational data with relative ease. This eliminated the degree of uncertainty that previously might have compromised the statistical evaluation of the hydrogeologic data.

GWMAP's success in integrating existing digital data to automate the well selection process clearly demonstrated the importance of the ability to share information with others and the great need for a broadly applied standard for data conversion and transfer.

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Acknowledgments

The authors wish to thank Renee Johnson of the Minnesota Department of Natural Resources for her work to convert the PLS data layer to GIS coverage. Susanne Maeder of LMIC supplied the statewide CWI coverage, and Susan Schreifels of MPCA conducted research on the LDP and made valuable suggestions on implementing GPS.

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Use of GIS in Modeling Ground-Water Flow in the Memphis, Tennessee, Area

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Abstract

Memphis, Tennessee relies solely on ground water for its municipal and industrial water supply. Memphis Light, Gas, and Water (MLGW) Division owns and operates over 160 water wells in 10 production fields throughout Shelby County. MLGW produces an average of approximately 200 million gallons per day, excluding much of the industrial demand. The city obtains its water from a thick, prolific aquifer known as the Memphis Sand, which was thought to be separated from a surficial aquifer by a thick confining layer. In recent years, evidence of leakage from the surficial aquifer to the Memphis Sand has been found.

The University of Memphis Ground Water Institute (GWI) is developing a hydrogeologic database of the Memphis area to study the aquifer. The database serves as the basis for several ground-water flow models that have been created as well as part of the wellhead protection programs currently being developed for Memphis and other municipalities in Shelby County. A geologic database was developed and is constantly being updated from borehole geophysical logs made in the area. Well locations are being field verified using a global positioning system (GPS).

Use of the database has allowed the development of a three-dimensional model of the Memphis area subsurface. The database also contains locations of and information on both private and public production and monitoring wells, Superfund sites, underground storage tanks, city and county zoning, land use, and other pertinent information. Procedures for linking the database to ground-water flow and solute transport models have been developed. The data visualization capabilities and the ability to link information to geographic features make geographic information systems (GIS) an ideal medium for solving ground-water problems.

An example of GIS use in ground-water flow modeling is the study of the Justin J. Davis Wellfield. The water quality parameters of alkalinity, hardness, sulfate, and barium have significantly increased over the past 10 years at this facility. To understand why these changes are occurring, MLGW, the GWI, and the U.S. Geological Survey (USGS) participated in a joint investigation of the wellfield.

In the spring of 1992, a series of 12 monitoring wells was drilled into the surficial aquifer near the production wells. Geophysical logging and split-spoon sampling revealed an absence of the confining layer, referred to as a window, at one of the monitoring wells. All other wells penetrated various thicknesses of clay. This window in the confining layer suggests that the water quality changes could be due to leakage from the surficial aquifer to the Memphis Sand.

The GIS database was used to construct a flow model of the Davis area. Also, using the surface modeling capabilities of GIS, the extent of the confining layer window was estimated and used to calculate leakage between the two aquifers. The results of these analyses also indicate that further subsurface exploration is needed to more accurately define the extent of the confining layer window.

Introduction

Memphis, Tennessee, relies solely on ground water for its municipal and industrial water supply. The Memphis Light, Gas, and Water (MLGW) Division owns and operates over 160 water wells in 10 production fields throughout Shelby County, as shown in Figure 1. MLGW produces an average of approximately 200 million gallons per day, excluding much of the industrial demand.

The city obtains its water from a thick, prolific aquifer known as the Memphis Sand, which was thought to be separated from the surficial aquifer by a thick confining layer. In recent years, evidence of leakage from the surficial aquifer to the Memphis Sand has been found. The University of Memphis Ground Water Institute (GWI) is developing a hydrogeologic database of the Memphis area to study the aquifer. Several ground-water

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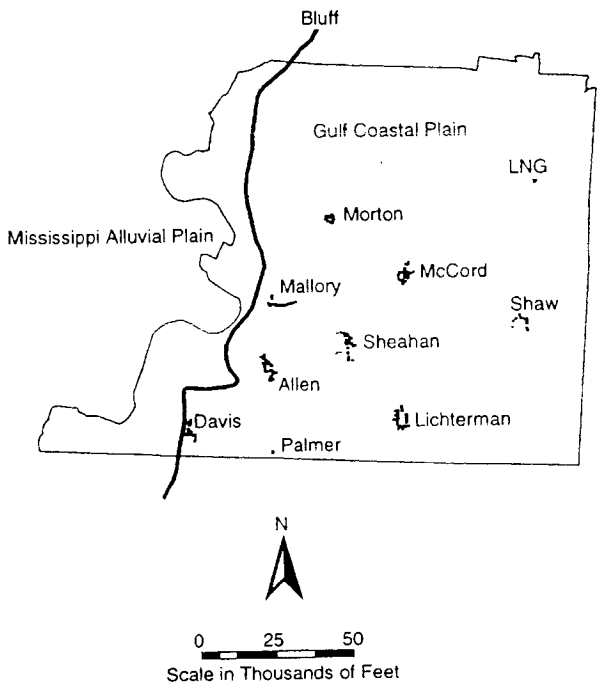

Figure 1. Physiographic description and MLGW Wellfields in Shelby County.

flow models have been developed using the database. Also, the database is an integral part of wellhead protection programs being developed for Memphis and other municipalities in Shelby County. A geologic database was developed and is constantly being updated from borehole geophysical logs made in the area. Well locations are being field verified using a global positioning system (GPS).

Use of the database has allowed the development of a three-dimensional model of the subsurface of the Memphis area. The database also contains locations of and information on both private and public production and monitoring wells; Superfund sites; underground storage tanks; city and county zoning; land use; and other pertinent information. Water quality measurements for every MLGW production well have been obtained, and a history of water quality for the Memphis Sand is being developed. Procedures have been developed for linking the database to ground-water flow and solute transport models (1). The data visualization capabilities and the ability to link information to geographic features make geographic information systems (GIS) an ideal medium for solving ground-water problems.

GIS Database

The GWI has developed and is continuing to update a hydrogeologic database for the Memphis area. ARC/INFO (marketed by Environmental Systems Research Institute, Redlands, California) is the GIS program that the GWI is using. The program runs on a network of 10 SUN SPARC stations. The capabilities of ARC/INFO and the computational speed of the SPARC stations allow very sophisticated ground-water analyses to be performed and have allowed the development of an extensive electronic database.

The basic unit of data storage in ARC/INFO is a coverage. A coverage is a digital representation of a single type of geographic feature (e.g., points may represent wells, lines may represent streets or equipotential lines, and polygons may represent political boundaries or zoning classifications). Information may be associated with an individual geographic feature in a feature attribute table. This information may then be queried and used in analyses. ARC/INFO also has its own macro language that allows the customization and automation of many ARC/INFO procedures.

A relatively new feature of ARC/INFO is address matching. This procedure compares a file containing the street address of a particular feature with an address coverage. This coverage is basically a library of addresses that are linked to a geographic coordinate. As the addresses from the input file are compared with the address coverage, the matching points are written to a second coverage. Any addresses in the input file that do not match an address in the address coverage are written to a "rejects" file. These can be matched by hand on a one-by-one basis.

Address matching serves as an alternative to digitizing, as long as a good address coverage for a specific area exists. The GWI has used this capability extensively and has developed a coverage of underground storage tank (UST) locations inside Shelby County. A database of private and monitoring wells is also being developed and updated using address matching. The raw information was obtained in an ASCII format from the appropriate regulating agencies (i.e., the Tennessee Department of Environment and Conservation Division of Underground Storage Tanks and the Memphis/Shelby County Health Department). The ASCII information was imported into ARC/INFO and address matched. The creation of a suitable address coverage and completion of the address matching of the UST file has taken almost 2 years. The private well coverage is currently being updated from historical information provided by regulating agencies and local well drilling companies.

An important part of the database is the geologic information obtained from geophysical logs in the area. Gamma logs, resistivity logs, and spontaneous potential (SP) logs are three major types of electric geophysical logs. Gamma logs measure naturally occurring radiation emitted from soil in the borehole. Clays and shales emit gamma rays. A high gamma count indicates the presence of clay or shale, and a low gamma count implies that little or no clay is present. Sand layers that contain fresh water are located using resistivity logs. Maximum values of resistivity indicate the possibility of a sand

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layer. Clays and sands that contain salt water may exhibit similar resistivities. SP logs are used to differentiate between the two (2).

Corroborating data such as formation logs, geologic studies, and available material samples should be consulted when reading and interpreting geophysical logs. The accuracy and reliability of an application based on well logs is completely dependent on a realistic interpretation of the geophysical data. A sample interpretation of a set of geophysical logs is shown in Figure 2. The results of interpretations like this are entered into the point attribute file of a well coverage.

The Triangulated Irregular Network (TIN) module of ARC/INFO is used to create a three-dimensional surface from information stored in a coverage. TIN creates a surface from a set of nonoverlapping triangles defined by a set of irregularly or regularly spaced points. In this study, the points defining the triangular TIN surfaces are the locations of the wells in the model area. TIN uses various interpolation routines to estimate surface values. Once the surfaces have been developed, two-dimensional profiles can be made that show the relative thicknesses of the various soil strata, as shown in Figure 3. These profiles aid in selecting boundary conditions and defining layers in ground-water flow models (3, 4).

In addition to the creation of profiles, a process that extracts surface values for use in a ground-water flow model has been developed. The GWI uses the United States Geological Survey (USGS) flow model, MODFLOW (5). Being a cell-based model, MODFLOW requires a value for each hydrogeologic parameter for each cell in the model grid. A series of FORTRAN programs and arc macro language (AML) programs were coupled to extract the required hydrogeologic data from surface models. For example, piezometric surface values are required to set initial conditions for the model. A coverage of the piezometric surface of the Memphis Sand was created, converted to a TIN surface, and the required values for each cell in the model were extracted using the procedure described above.

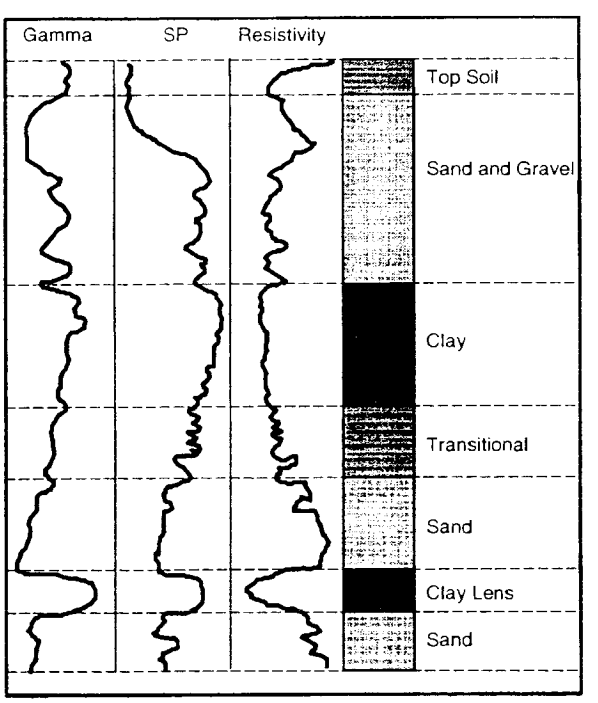

Figure 2. Example interpretation of geophysical logs.

The results and hydrogeologic data from the calibrated model can be read back into the database and converted into coverages. This allows piezometric contours to be developed and displayed with other information in the database to aid in decision-making. Also, capture zones for the wells can be brought into the database and compared with surface features like industries, landfills, Superfund sites, UST locations, or other sites that may have an impact. This has proved especially helpful in developing wellhead protection programs where a complete contaminant source inventory must be performed for the capture zone of each well and within a fixed radius around the well. The procedure used to develop model data from the GIS database is summarized in Figure 4.

McCord Wellfield Wellhead Protection Program

MLGW and the GWI performed a demonstration project funded by the U.S. Environmental Protection Agency (EPA) for the C.M. McCord Wellfield Wellhead Protection Program (6). This wellfield was selected because of multijurisdictional problems that will be encountered during plan implementation. The City of Memphis owns all the wells, but many of them, and all future well lots, are located within the city limits of Bartlett. A wellhead protection plan will have to involve the cooperation of both municipalities. The existing wellfield is shown in Figure 5.

Tennessee wellhead protection regulations require the delineation of two zones of protection for a city the size of Memphis: a 750-foot radius around the wellhead and a 10-year capture zone for the well. The 10-year capture zone (called the Zone 2 area) was delineated using two flow models and information obtained from the GWI database. Results were imported into the GIS database and compared with existing information. The Zone 1 area was delineated by buffering each well point in the coverage by the appropriate radius. Each well location was verified using a Trimble GPS unit and is accurate to within 2 meters. A contaminant source inventory was performed using the coverages developed by address matching. The primary potential sources of contamination in this area are USTs. A windshield survey located other potential sources, such as dry cleaners. These locations were entered into the database also by using

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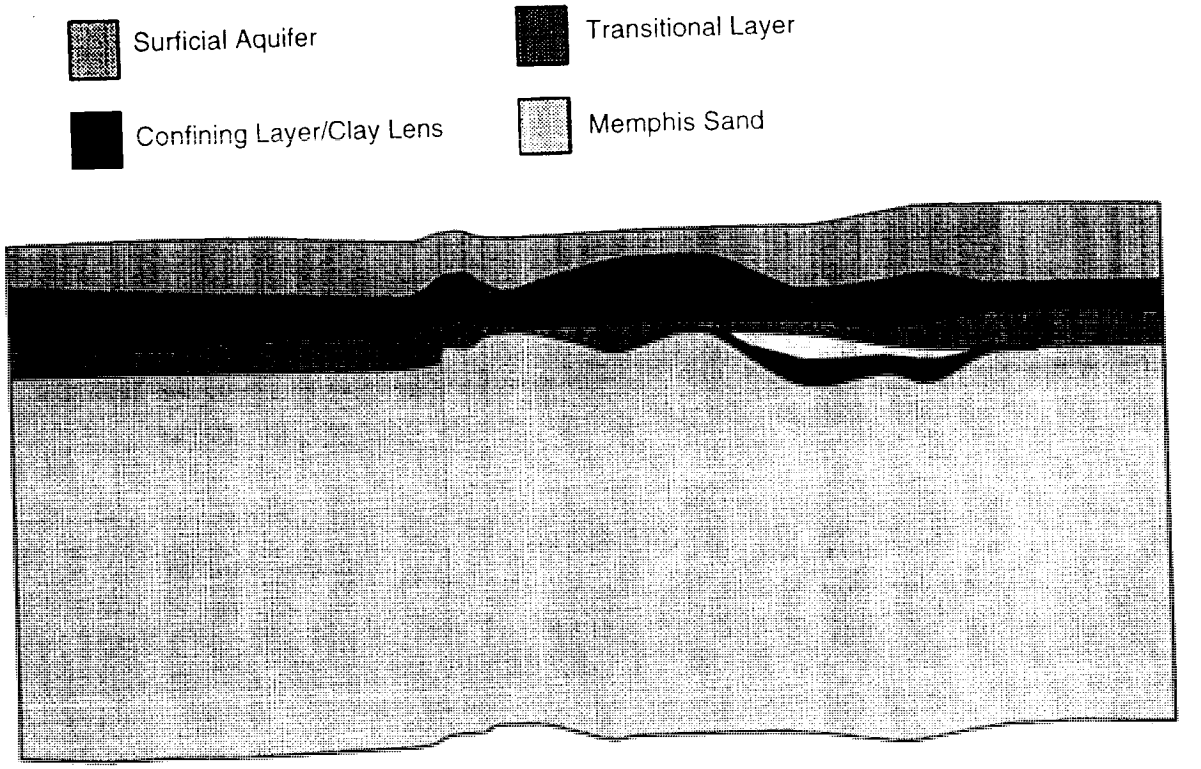

Figure 3. Two-dimensional profile constructed from surface models.

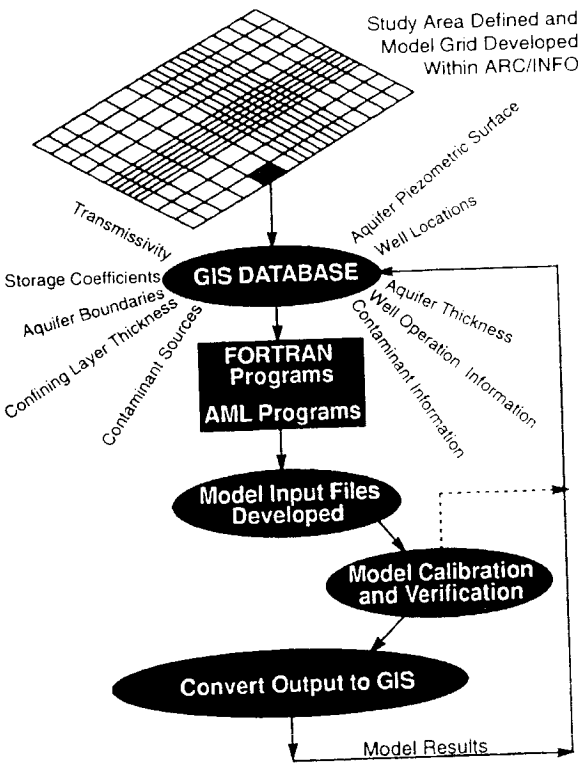

Figure 4. Procedure for integrating GIS and flow model.

the address-matching capabilities of ARC/INFO. The Zone 2 areas for present wells and future wells, along with the potential sources of contamination, are shown in Figure 6.

Davis Wellfield Study

An example of GIS use in ground-water flow modeling is the study of the Justin J. Davis Wellfield. The Davis Wellfield is one of 10 producing fields operated by MLGW. It is located in the southwestern corner of Shelby County and consists of 14 wells, as shown in Figure 1. Production at the Davis Wellfield began in 1971, and an estimated 13 million gallons per day are currently withdrawn from the Memphis Sand aquifer. Since 1972, MLGW has collected water quality data from the wells at the Davis Wellfield, including values for alkalinity, hardness, chloride, sulfate, iron, and barium. Water quality parameters of alkalinity, hardness, sulfate, and barium have significantly increased in the past 10 years.[1] A possible explanation for the change in water quality is water leakage from the upper aquifer through the confining unit to the Memphis Sand aquifer.[1] The water chemistry from the two aquifers is noticeably different. The surficial aquifers generally have a higher total dissolved solids concentration, hardness, and alkalinity than water from the Memphis Sand.[1]

[1] Webb, J. 1992. Memphis Light, Gas, and Water Division. Personal interview.

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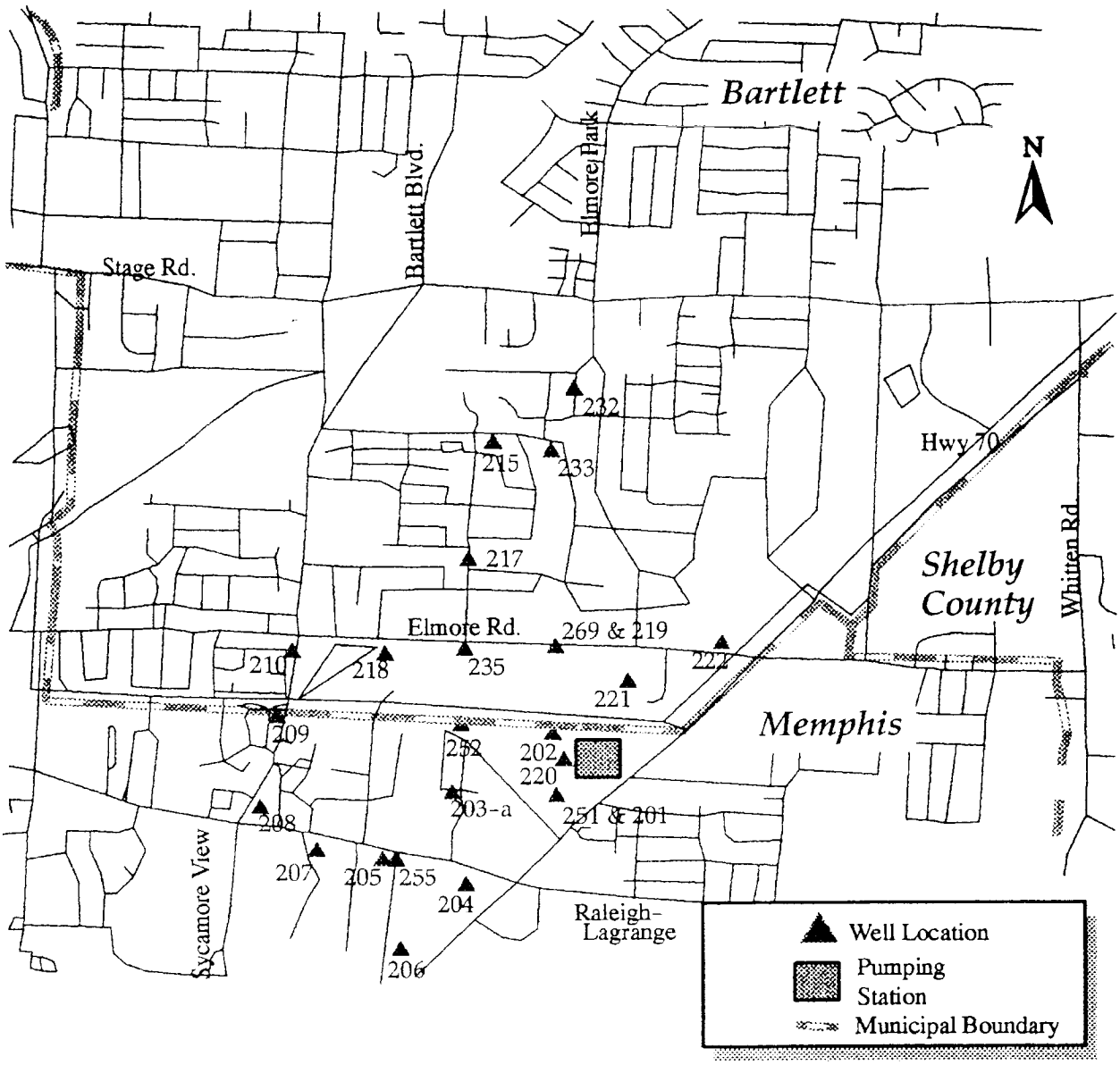

Figure 5. Existing McCord Wellfield.

MLGW, the GWI, and the USGS participated in a joint investigation of the wellfield to determine why the water quality changes are occurring. In the spring of 1992, 12 monitoring wells were drilled into the surficial aquifer near the production wells, as shown in Figure 7.

Geophysical logging and split-spoon sampling revealed an absence of the confining layer at one monitoring well, GWI-3. All other wells penetrated various thicknesses of clay. This "window" in the confining layer suggests that the water quality changes could be due to leakage from the surficial aquifer to the Memphis Sand. The logs for these monitoring wells were combined with an existing geophysical log coverage. The extent of the confining layer window was estimated using GIS surface modeling capabilities, as shown in Figure 8.

Two-dimensional profiles were created to further show the extent of the confining layer window. The locations of the profiles in relation to various surface features are shown in Figure 9. Profiles 1 and 2 were taken across the river bluff, and Profiles 2 and 3 were taken across the window. The profiles are shown in Figure 10.

Many important features of this area's geology can be inferred by looking at the profiles. A connection of the alluvial and fluvial aquifers is shown in Profile 2. Elsewhere along the bluff, the connection of the two aquifers is less prominent, as shown in Profile 1. The connection of the two aquifers in Profile 2 may be the cause of a peculiar mounding effect in the water table of the alluvial aquifer in that area. The thinning of the top soil in Profiles 1 and 3 may indicate a local recharge area for the

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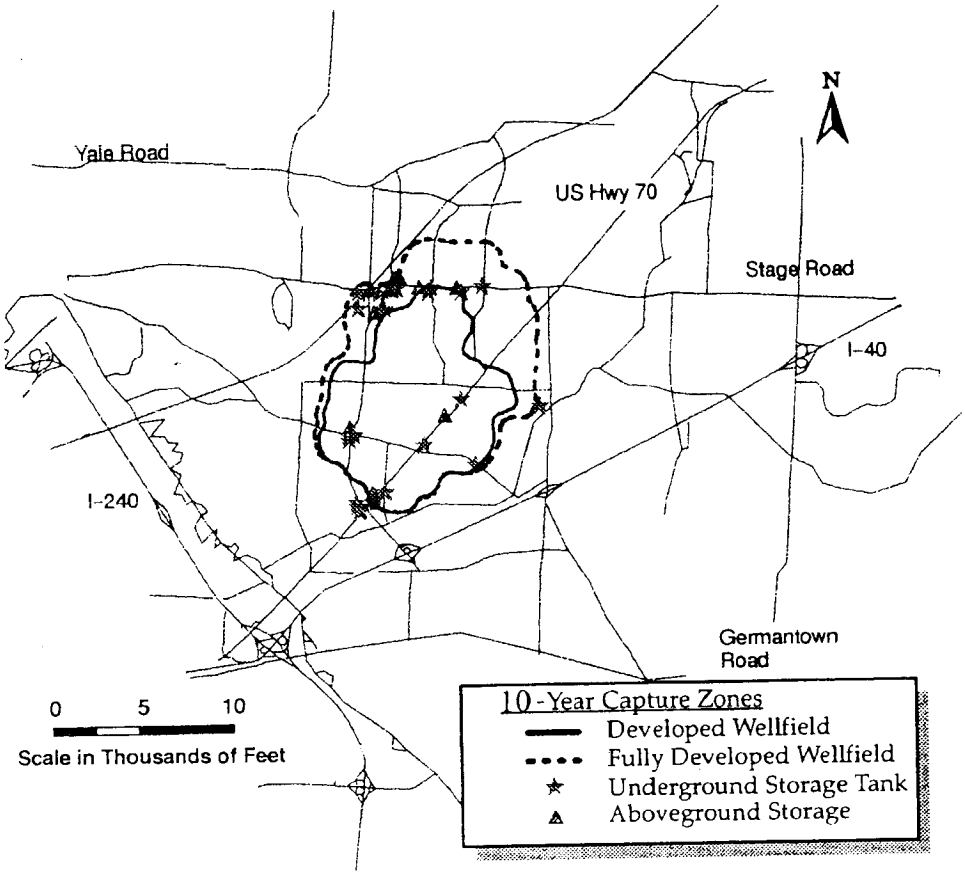

Figure 6. Underground storage tanks and aboveground storage locations.

alluvial aquifer. Profiles 2 and 3 show the confining layer window that suggests a connection between the surficial aquifer and the Memphis Sand.

Following the convention of the USGS, a "window" is defined as any area where the aggregate clay thickness is less than 10 feet (8). A surface of the thickness of the confining layer was generated from the geophysical log coverage. The surface model was converted to a contour line coverage on a 5-foot interval. Using the ARC-EDIT module of ARC/INFO, the contour line coverage was converted to a polygon coverage. The area bounded by the 10-foot contour of the surface model was calculated to be 840,000 square feet (about 19 acres). The area was calculated by adding the areas between the 10- and 5-foot contours and the area within the 5-foot contour.

A flow model of the area was developed based on the hydrogeologic data contained in the database. A steady state model was calibrated to hydraulic conditions recorded during fall 1992 by the USGS and the GWI; the root mean square (RMS) error for this model was 1.76 feet. A second steady state model was developed to simulate conditions recorded during spring 1993 (a period of high water levels in area lakes). The RMS error for this simulation was 5.19 feet. This higher error may indicate that the high water levels in surface water bodies are not realistic for steady state boundary conditions. Realistically, monitoring wells that are relatively far from a surface water body are affected more by average water levels over time rather than relatively short periods of highs and lows.

Using average values of h_1, h_2 (head in upper and lower aquifers), l (vertical flow distance), and *VCONT* (a parameter used in MODFLOW to allow for vertical conductance), the estimated flow rate through the window for fall 1992 may be computed as:

$$k = \text{VCONT } 1 = 1.76e^{-3} \times 199.2 = 0.351 \frac{\text{ft}}{\text{day}}$$

$$i = \frac{h_1 - h_2}{l} = \frac{186.5 - 156.1}{199.2} = 0.153$$

$$Q = kAi = 0.351 \times 840{,}000 \times 0.153 = 45{,}111 \frac{\text{ft}^3}{\text{day}}$$

$$45{,}111 \frac{\text{ft}^3}{\text{day}} \times 7.48 \frac{\text{gallons}}{\text{ft}^3} = 337{,}430 \frac{\text{gallons}}{\text{day}} = 0.34 \text{ MGD}$$

The flow rate calculated from average spring 1993 (a period of high water levels in the surficial aquifers) model results was computed as:

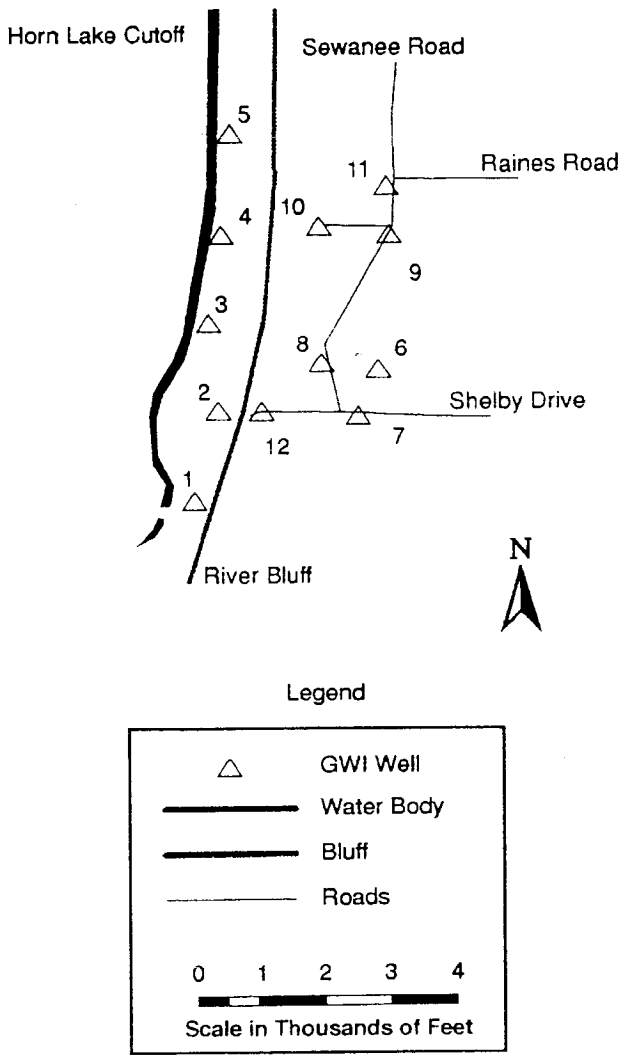

Figure 7. Location of GWI monitor wells.

$$k = VCONT\ l = 1.76e^{-3} \times 199.2 = 0.351\ \frac{ft}{day}$$

$$i = \frac{h_1 - h_2}{l} = \frac{196.4 - 156.8}{199.2} = 0.199$$

$$Q = kAi = 0.351 \times 840{,}000 \times 0.199 = 58{,}673\ \frac{ft^3}{day}$$

$$58{,}673\ \frac{ft^3}{day} \times 7.48\ \frac{gallons}{ft^3} = 438{,}874\ \frac{gallons}{day} = 0.44\ MGD$$

Using the GIS-delineated window, an estimated 0.34 to 0.44 million gallons per day flow from the alluvial aquifer to the Memphis Sand aquifer. This variation in the flow rate was due to seasonal variations of water level in the alluvial aquifer.

Since the wellfield pumps approximately 13 million gallons per day, the effect of the window on the entire Davis Wellfield may not be significant. The window lies within the 30-year capture zone of two wells in the field, however.

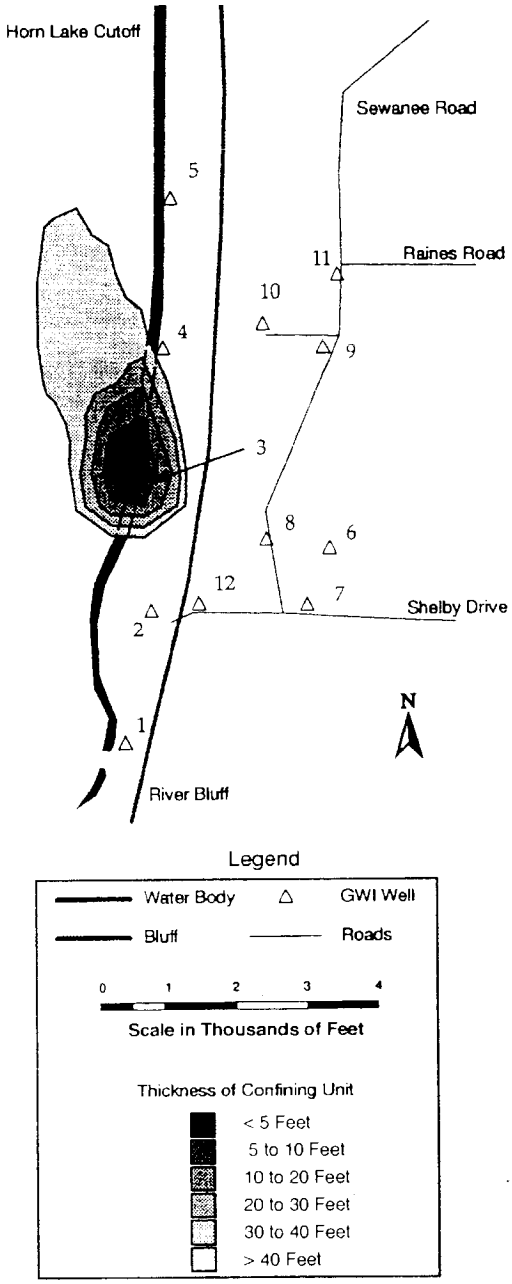

Figure 8. Location and extent of window in confining unit.

The flow rate through the window is approximately 20 percent of the total production of these two wells. Additionally, since the wells would probably not operate simultaneously, the flow rate through the window may account for approximately 40 percent of the flow at either well.

Particle tracking in the Memphis Sand was developed using MODPATH (9) from MODFLOW results. Particles were placed at model well screens and tracked backward for 30 years. The output from MODPATH was read into the GIS database for comparison with other data, as shown in Figure 11. The hole in the confining unit lies

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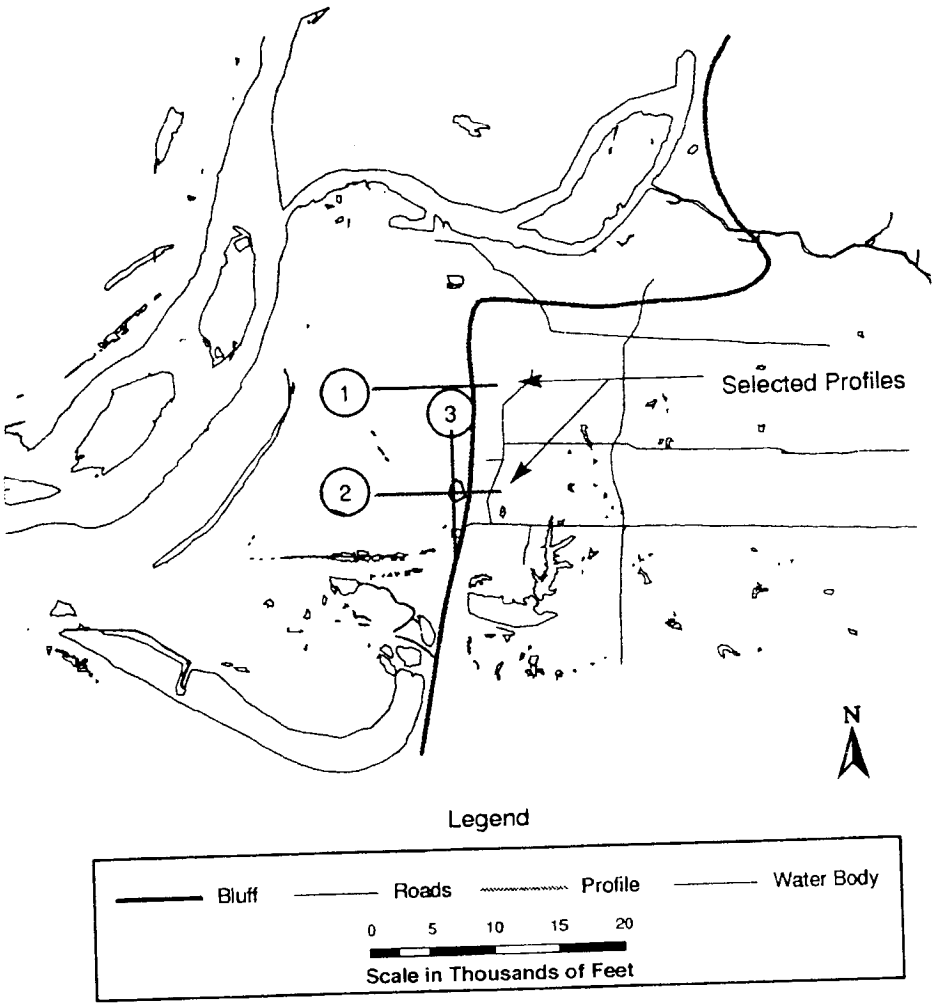

Figure 9. Location of selected profiles.

within the 30-year capture zone of MLGW wells 418 and 419. Historically, 419 was the first well that experienced water quality changes. The water quality is becoming similar to water found in the alluvial aquifer.

A change in the water quality in well 418 is not as immediately noticeable as in well 419. This inconsistency in data may indicate that the window does not extend northward from GWI-3, as the TIN model predicted. To determine which capture zone (418 or 419) encompasses GWI-3, particles were tracked backward for 40 years, as shown in Figure 12. GWI-3 lies on the edge of the capture zone for 419. The flow lines from 418 and 419 move toward the northwest and southwest in the Memphis Sand, pass up through the window, and emerge in the upper aquifer.

Conclusions

The explanation of the Davis Wellfield investigation addressed some limitations of the database. The utility that this hydrogeologic database provides greatly outweighs the disadvantages, however. Without the ability to map and define hydrogeologic features, this project may not have been completed in the allotted time frame or may not have been completed in the same level of detail. GIS greatly enhances the development and evaluation of ground-water flow models.

Specific conclusions that can be drawn from the analysis performed in this project are:

- The delineation of a window in the confining layer using a GIS database is possible.
- Based on the GIS-generated window, an estimated 0.34 to 0.44 million gallons per day flow from the upper aquifer to the Memphis Sand, which may account for as much as 40 percent of the flow at either well 418 or 419.
- The drilling of more monitoring wells north, south, east, and west of GWI-3 may provide for a more accurate delineation of the window.

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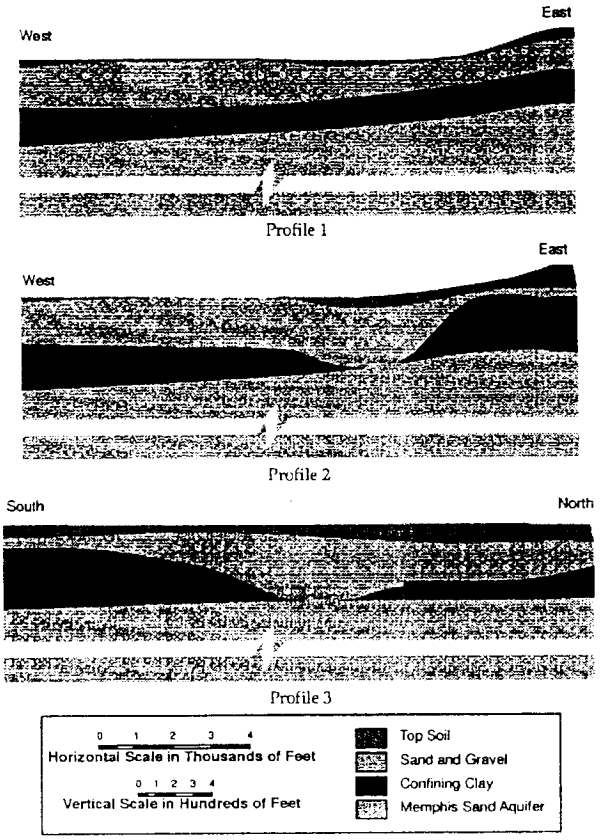

Figure 10. Selected subsurface cross sections in the Davis Wellfield.

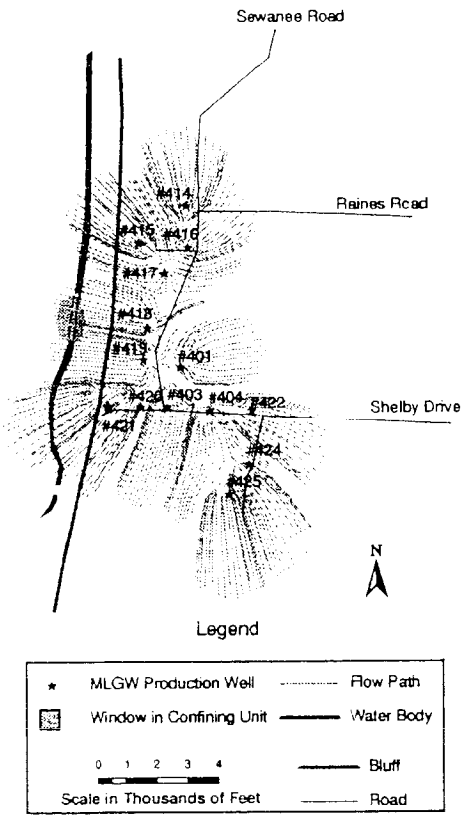

Figure 11. Backward tracking for 30 years.

General conclusions that may be drawn from this discussion are:

- GIS provides a convenient method of viewing flow model input and output.
- A flow model may be developed and evaluated in a relatively short time using GIS.
- GIS provides a convenient means of compiling and managing the information required to develop a wellhead protection program.

Some GIS disadvantages that have been noted are:

- The time required to develop a database and learn to apply the GIS program in a particular situation may be prohibitive.
- GIS-generated results from a limited database may be misleading and should be corroborated with other analysis methods.

Acknowledgments

The authors would like to acknowledge the efforts of the professors and students of the GWI, both present and past, for their contributions to the database and this

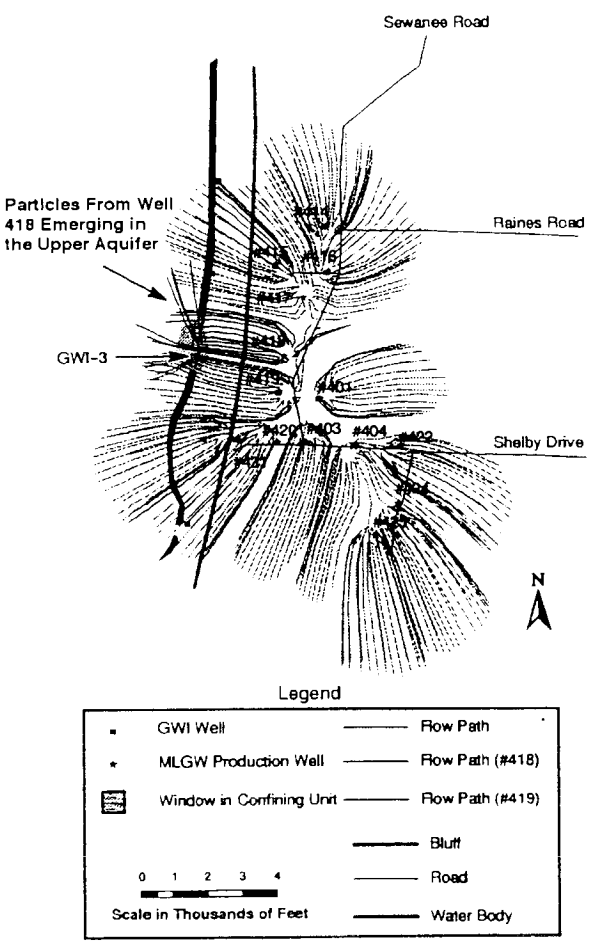

Figure 12. Backward tracking for 40 years.

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project, especially: Dr. John W. Smith, Director, and Dr. Charles V. Camp for their patience and guidance in interpreting the model results and hydrogeologic conditions of the area; and David W. Kenley, Brian A. Waldron, and Robert B. Braun for their help in creating the well log database.

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MODRISI: A PC Approach to GIS and Ground-Water Modeling

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Abstract

It is widely accepted that ground-water contamination problems cannot be adequately defined or addressed until the governing physical, chemical, and biological processes affecting the transport and fate of contaminants are adequately characterized. Recent research has led to a better understanding of these complex processes and their effect on the movement of contaminants in the subsurface. The compilation and application of such information has yet to be accomplished at many hazardous waste sites, however. Too often, copious quantities of data are collected, only to be stored, ignored, or misplaced, rather than used for problem-solving. Geographic information systems (GIS) are computer-based tools that are relatively new to many environmental professionals. GIS allows the manipulation, analysis, interpretation, and visualization of spatially related data (e.g., hydraulic head, ground-water velocity, and contaminant concentration). GIS is more than a cartographic utility program, however. The analytical capabilities of GIS allow users to display, overlay, merge, and identify spatial data, thereby providing the basis for effective environmental decision-making.

IDRISI is a widely used PC-based raster GIS system that provides numerous analytical capabilities that are directly applicable to hydrogeologic studies. Raster systems are particularly well suited for analysis of continuous data such as elevation (e.g., water table, land and bedrock surfaces), precipitation, recharge, or contaminant concentrations and may be readily integrated with finite-difference ground-water models. Because the formats for IDRISI and ground-water model input data sets are different, a need exists for a program to integrate these two types of robust tools.

MODRISI is a collection of utility programs that allows easy manipulation and transfer of data files between IDRISI and ground-water models (e.g., MODFLOW, ASM, MOC). In addition, MODRISI integrates other widely used commercial and private domain software packages, such as SURFER, Geopack, GeoEas, AutoCAD, CorelDraw, and various spreadsheet programs. Two-dimensional arrays of models' input data sets can easily be created from IDRISI image files. AutoCAD vector files obtained by digitizing model boundaries, well locations, rivers and streams, or U.S. Geological Survey digital elevation model (DEM) files can also be translated into model input file formats. MODRISI can process model output files and prepare GIS image files that can be displayed and manipulated within IDRISI. Thus, MODRISI is more than a pre- and postprocessor for ground-water models; it is a complete GIS/ground-water modeling interface that is accessible to most ground-water hydrologists.

Introduction

Hydrogeologists collect and analyze large volumes of data during a ground-water modeling process. These data are stored and presented in many different forms such as maps, graphs, tables, computer databases, or spreadsheets. To most hydrogeologists, geographic information systems (GIS) are relatively new tools. They have been developed and applied in other natural and social science fields for over two decades, however, and can also be used in the ground-water modeling process.

GIS represents a new, powerful set of tools that can significantly improve the usefulness of results obtained during the ground-water modeling process. Bridging the disciplines of ground-water modeling, computer graphics, cartography, and data management, GIS represents a computer-based set of tools to display and analyze spatial data (e.g., water level elevations, ground-water quality data, modeling results, ground-water pollution potential). Efficient use of increasingly large volumes of

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data can be achieved only with powerful systems capable of acquiring information from a variety of sources, scales, and resolutions.

GIS can be defined as a computer-assisted system for the efficient acquisition, storage, retrieval, analysis, and representation of spatial data. Most GIS platforms consist of numerous subsystems that perform the listed tasks. The subsystems have the ability to query spatially related information and incorporate statistical analyses and modeling of relations and their temporal changes within the database. More than just a mapping system, GIS allows the user to analyze spatially related data and visualize results in either paper map form or graphically on screen. The data to be analyzed are a collection of spatial information represented by points, lines, and polygons and their associated attributes (characteristics of the features such as elevation or concentration). The cartographic tools of GIS allow the analyst to display, overlay, measure, merge, and identify the data to support a particular analysis. By allowing spatial data analysis and display, GIS provides the means necessary for effective environmental decision-making and implementation of environmental management plans.

GIS uses two basic map representation techniques: vector and raster. Vector representations describe features with a number of connected points. Raster representations subdivide a study area into a mesh of grid cells, each cell containing either a quantitative attribute value or feature identifier. Raster systems are well suited for analysis of continuous data (e.g., water level elevations, infiltration and recharge rates). This makes raster-based systems ideal for integration with ground-water models that use regularly spaced nodes. The objective of this paper is to illustrate such an integration of MODFLOW (1), a widely used U.S. Geological Survey (USGS) finite-difference ground-water flow model, and IDRISI (2), a raster-based GIS.

Previous Studies

To enhance understanding of a hydrogeologic system, and also to develop a credible ground-water model of the system, hydrogeologic features such as lithological logs, recharge and withdraw rates, estimates of spatial distribution of hydraulic conductivity, or specific storage can be plotted using GIS capabilities of data retrieval and overlay options to interactively define an area of interest (3-6). Two previous studies that combined GIS and ground-water modeling are briefly described.

Torak and McFadden (7) used GIS to facilitate finite-element modeling of ground-water flow. Complex aquifer geometry and irregularly distributed aquifer-system characteristics that influence ground-water flow affect the design of the finite-element mesh. GIS systems represent the complex arrangement of nodes and elements and the distribution of aquifer properties to provide input to the flow model. Point-data coverages of pertinent aquifer characteristics are rated from a relational database and are displayed using GIS.

Contoured surfaces based on point-data coverages produce triangulated irregular networks (TINs) that are superposed on the finite-element mesh to delineate zones of elements having similar aquifer properties. Zone boundaries are identified using the contoured TIN surface and by manually determining where boundaries align with the element sides. The allocation of well pumping rates to nodes in the finite-element mesh is performed efficiently with GIS for model input. Well pumping rates are accumulated by element from the combined coverages of the pumping data and the mesh, and element data are distributed to the node points for input. GIS is also used to prepare data for model input and to assess the adequacy of the data prior to simulation.

Three-dimensional perspectives showing TIN coverages of aquifer-property data are used to analyze and interpret complexities within the flow system before zonation. Additionally, GIS is used to display computed hydraulic heads over the finite-element mesh to produce contour maps of the simulated potentiometric surface. Because the node points in the finite-element mesh are not arranged in an orthogonal fashion, such as a finite-difference grid, a map display of the computed values of hydraulic head at the nodes is prepared for efficient and accurate interpretation of simulation results.

Harris et al. (8) conducted the Remedial Investigation/Feasibility Study (RI/FS) of the San Gabriel basin. Vast amounts of hydrogeologic data have been gathered, and a comprehensive systematized GIS database has been developed. The identified hydrologic boundaries, recharge basins, stream locations, well locations, and contaminant distributions are some of the features considered in developing a base map. The GIS-generated base map has allowed development of a finite-element grid for the basin. For each finite element, the initial estimates of the hydraulic conductivity, specific yield, recharge rates, and other input parameters were provided.

Using simple interfacing programs, the retrieval GIS nodal and elemental data were converted to required formats for the input files of the Couple Fluid, Energy, and Solute Transport (CFEST) code. Simulated ground-water levels were compared with the GIS-generated potentiometric surfaces. In areas of wide variations between simulated and observed data, the zonal distribution of controlling parameters was reevaluated, analyzed, and updated. Data processing, development of input files for computerized analysis of ground-water flow, and analysis of simulation results with different alternative conceptualizations is time consuming and tedious. Efficient use of GIS and CFEST not only eased the burden of conducting multiple simulations but

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reduced the probability of errors as well as the amount of time and effort required for each simulation.

IDRISI

IDRISI is a grid-based geographic information and image processing system developed by the Graduate School of Geography at Clark University and supported by the United Nations Institute for Training and Research (UNITAR) and the United Nations Environment Programme Global Resource Information Database (UNEP/GRID) (9). IDRISI is a collection of over 100 program modules that are linked through a menu system. These programs are organized into several groups:

- The core modules provide data entry and database management capabilities.
- The geographic analysis modules provide tools for database analysis.
- The statistical analysis modules allow statistical characterization of images.
- The peripheral modules provide a series of utilities.

IDRISI and other raster-based systems divide data sets into map layers; each layer contains data for a single attribute. For the example of a ground-water model, these layers could correspond to the MODFLOW two-dimensional arrays (e.g., initial water levels, transmissivity distribution, IBOUND arrays, computed hydraulic heads). IDRISI provides many analytical tools that are useful in hydrogeologic studies.

Three of the most important categories of these tools are database query, map algebra, and context operator. A semihypothetical case described below illustrates the use of these analytical tools. IDRISI provides an extensive set of tools for image processing, geographic and statistical analysis, spatial decision support, time series analysis, data display, and import/export and conversion. In addition, as a set of independent program modules linked to a broad set of simple data structures, the system is designed such that researchers may readily integrate into the system their own modules, written in any programming language.

IDRISI uses three types of data files: image, vector, and attribute. Image files contain rasterized information relating to a spatial variable. Vector files contain the coordinates of points, lines, and polygonal features. An attribute file lists the identifiers of features and the associated attribute values. Values files can be extracted from the existing image files, or image files can be created from existing values files. The values files can be combined and stored in a dBASE format. Each image, vector, or attribute file has a corresponding documentation file that contains information about the data file (e.g., title, number of rows and columns).

MODRISI: MODFLOW/IDRISI Interface

MODRISI is a set of utility programs that allows the transfer of data files between MODFLOW, IDRISI, Golden Software SURFER, GeoEas, and other software. Preparation of two-dimensional arrays for the MODFLOW input files is generally tedious and time consuming. The arrays can be created easily from the IDRISI image files, however. Thus, MODRISI serves as a preprocessor for MODFLOW. For example, when the values of a variable are available only for irregularly spaced points, interpolation routines in SURFER or GeoEas may be used to estimate the values of the variable on regularly spaced grid-nodes. MODRISI translates SURFER or GeoEas files into IDRISI image files for manipulation, analysis, and display. IDRISI recognizes either latitude and longitude geodetic coordinates or arbitrary Cartesian plane coordinates. IDRISI assigns the lower left grid-block of a raster image as a zero-row, zero-column block.

Vector files, such as model boundaries, well locations, and rivers, may be created within IDRISI and translated into a MODFLOW input file format. For example, the location of a river may be digitized on screen in IDRISI. The vector-to-raster function may be invoked, assigning all blocks through which the river passes as river nodes. Similarly, the positions of wells may be digitized and translated into the row-column positions and saved as a MODFLOW input file for the well package. Once the MODFLOW input files are prepared, MODFLOW simulations may be initiated. The MODFLOW hydraulic head output files may be read by MODRISI and modified to create IDRISI image files. Again, the image files may be displayed and evaluated within IDRISI. Thus, MODRISI is used as a postprocessor for MODFLOW.

Case Study

The utility of MODRISI was demonstrated at a hazardous waste site. Previous investigations provided site-specific information, including water level and bedrock and land surface elevations, which was then analyzed using GeoEas, a public domain geostatistical software program. These data were kriged to produce a grid of regularly spaced data. These data were imported into MODRISI and converted to IDRISI image files. IDRISI was used to visualize the surfaces that GeoEas generated.

Several prominent features are obvious upon inspection of the kriged bedrock topography (see Figure 1). A bedrock ridge trending northwest to southeast is flanked by a minor trough to the east and a major trough to the southwest. The outline of the site is visible on all figures. The kriged water level elevation map (see Figure 2) illustrates the general hydraulic gradient to the west. Land surface elevations (see Figure 3) range from greater than 200 feet in the northeastern portion of the site to a low of 166 feet on the western boundary.

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Figure 1. Bedrock elevation contour map derived from kriged data and transformed by MODRISI into IDRISI image file format.

Figure 2. Water level contour map derived from kriged water level data and transformed by MODRISI into IDRISI image file format.

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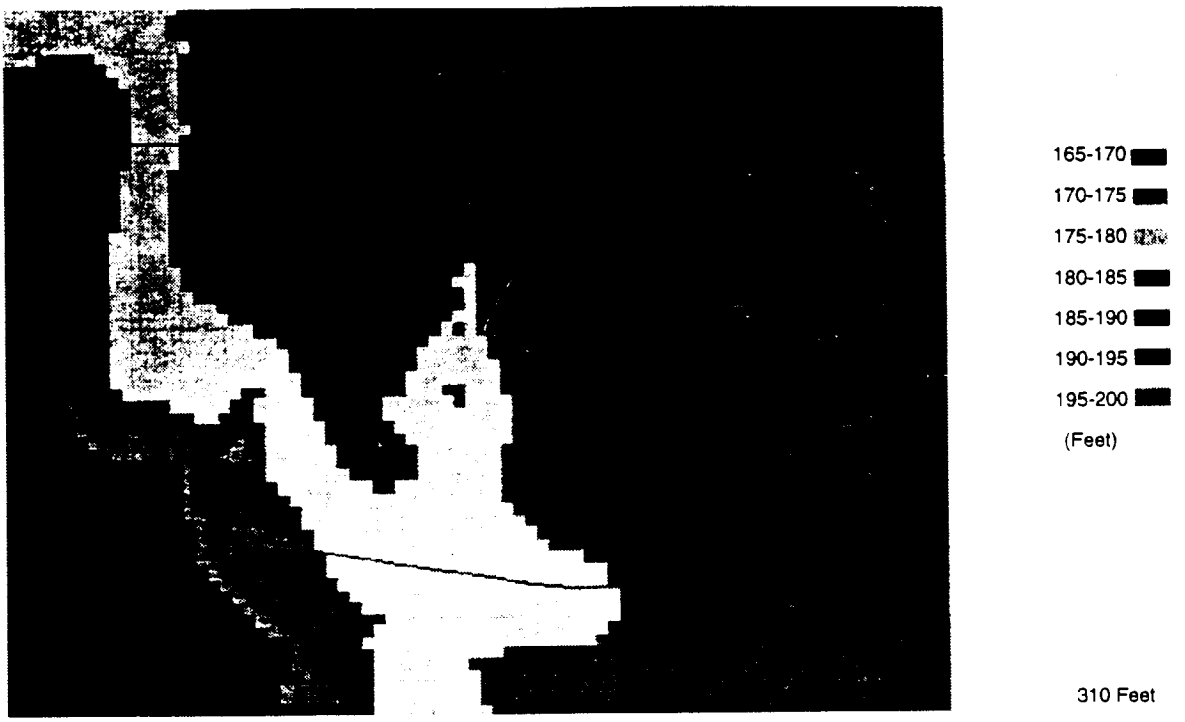

Figure 3. Land elevation contour map generated from kriged data and transformed by MODRISI into IDRISI image file format.

Additionally, the value (e.g., elevation) and (x,y) coordinates may be queried for any point, line, or area of an image file.

The analytical capabilities of IDRISI are illustrated in the following example. The saturated thickness was determined by subtracting the bedrock surface from the water level surface. The results illustrate the spatial variability of saturated thickness of the overburden aquifer (see Figure 4). The results correspond favorably with the general bedrock topography, as would be expected given the relatively low hydraulic gradient across the site. This OVERLAY (subtract) function may also readily be used to evaluate the adequacy of model calibration. For example, predicted values may be compared with observed values graphically, allowing the modeler to quickly visualize and identify areas of the model domain requiring additional consideration and manipulation.

Conclusions

One of the most tedious tasks in a ground-water modeling process is preparing input data and postprocessing model results. GIS allows rapid incorporation and evaluation of new site characterization information. The proposed combination of IDRISI, a raster-based GIS system, and MODRISI, a set of utility programs, could significantly reduce the amount of time necessary for entering data in required array formats. The visualization capabilities of IDRISI in conjunction with MODRISI and MODFLOW allow project managers to better understand the three-dimensional nature of subsurface environmental problems.

Acknowledgments

The authors wish to thank Chet Janowski, U.S. Environmental Protection Agency Region I, for his support on this project.

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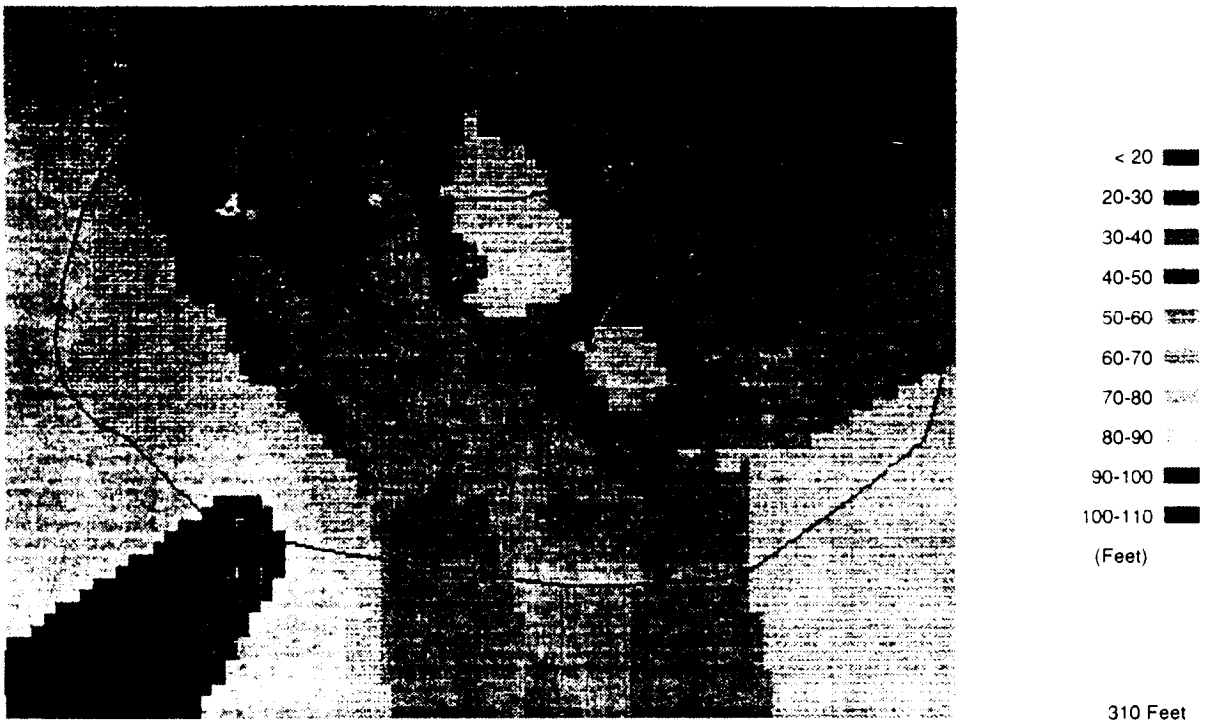

Figure 4. Saturated thickness contour map developed by subtracting bedrock image file from water level image file within IDRISI.

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GIS in Statewide Ground-Water Vulnerability Evaluation to Pollution Potential

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Abstract

The ground-water vulnerability of Indiana to pollution potential was evaluated using a geographic information systems (GIS) environment. The Geographic Resources Analysis Support System (GRASS) and the GRID submodule of ARC/INFO were used to conduct the analysis and to identify and display the areas sensitive to ground-water pollution potential. The state soils geographic (STATSGO) database was employed to retrieve statewide soils information required for the analysis. The information from the STATSGO database was used in two models, DRASTIC (acronym representing the following hydrogeologic settings: **D**epth to water table, aquifer **R**echarge, **A**quifer media, **S**oil media, **T**opography, Impact of vadose zone, and hydraulic **C**onductivity of the aquifer) and SEEPAGE (System for Early Evaluation of Pollution Potential of Agriculture Ground-Water Environments). These models employ a numerical ranking system and consider various hydrogeologic settings that affect the ground-water quality of a region. Ground-water vulnerability maps were prepared for the state of Indiana based on DRASTIC and SEEPAGE results. Continuing work is planned to determine the accuracy of the results by comparing the existing well-water quality data. The DRASTIC Index and SEEPAGE Index number (SIN) maps show great potential as screening tools for policy decision-making in ground-water management.

Introduction

Ground-water contamination due to fertilizer and pesticide use in agricultural management systems is of wide concern. In 1989, reports of ground-water contamination in New York wells led the U.S. Environmental Protection Agency (EPA) to conduct a nationwide survey on well contamination in the United States. These wells were tested for presence of nitrate, pesticides, and pesticide breakdown products (1). Statistically, the wells selected represent more than 94,600 wells in approximately 38,300 community water systems. Over 52 percent of the community water systems and 57 percent of the rural domestic wells tested contained nitrates (2).

Indiana has abundant ground-water systems providing drinking water for 60 percent of its population. A study on well-water quality detected pesticides in 4 percent of wells tested in Indiana. Also, 10 percent of private wells and 2 percent of noncommunity wells contained excessive nitrate levels (3).

Statewide maps showing the areas vulnerable to ground-water contamination have many potential uses such as implementation of ground-water management strategies to prevent degradation of ground-water quality and monitoring of ground-water systems. These maps will be helpful in evaluating the existing and potential policies for ground-water protection. Ground-water models such as SEEPAGE (System for Early Evaluation of Pollution Potential of Agriculture Ground-Water Environments) and DRASTIC (acronym representing the following hydrogeologic settings: **D**epth to water table, aquifer **R**echarge, **A**quifer media, **S**oil media, **T**opography, Impact of vadose zone, and hydraulic **C**onductivity of the aquifer) can be applied on a regional scale to develop such maps.

The data layers required for these models are commonly available data such as pH and organic matter content. For most states, the statewide ground-water vulnerability maps generated using DRASTIC were produced from 1:2,000,000-scale data (4). EPA (2) found that these maps did not correlate well with the water quality analysis performed for the national survey of pesticides in drinking water wells. States need more detailed and accurate maps to implement ground-water management programs. The state soils geographic (STATSGO) database at the 1:250,000-scale might be useful for studies at a larger scale.

The geographic information systems (GIS) environment is widely applied for diverse applications in resources management and other areas. It offers the facilities to store, manipulate, and analyze data in different formats

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and at different scales. The DRASTIC and SEEPAGE models can be integrated within the GIS environment to produce the final ground-water vulnerability maps.

Objectives

The purpose of the study was to prepare maps showing areas in Indiana vulnerable to ground-water pollution. This goal was accomplished by considering hydrogeologic factors in each region that affect the mobility and leaching of the contaminant reaching the aquifer. The prime objectives of this research were to:

- Evaluate Indiana's ground-water vulnerability to pollution potential using the DRASTIC and SEEPAGE models:
  - Integrate and evaluate the models in a GIS environment (Geographic Resources Analysis Support System [GRASS] ARC/INFO).
  - Develop a graphic user interface (GUI) in ARC/INFO to conduct the analyses.
- Compare the pollution potential map from the DRASTIC model with the map developed using the SEEPAGE Index number (SIN).
- Validate the accuracy of the present approach by comparing the vulnerability maps with the existing well-water quality data sampled across the state.

DRASTIC

DRASTIC is a ground-water quality model for evaluating the pollution potential of large areas using the hydrogeologic settings of the region (4-6). EPA developed this model in the 1980s. DRASTIC includes different hydrogeologic settings that influence a region's pollution potential. A hydrogeologic setting is a mappable unit with common hydrogeologic characteristics. This model employs a numerical ranking system that assigns relative weights to parameters that help evaluate relative ground-water vulnerability to contamination.

The hydrogeologic settings that make up the acronym DRASTIC are:

- *[D] Depth to water table:* Compared with deep water tables, shallow water tables pose a greater chance for the contaminant to reach the ground-water surface.
- *[R] Recharge (net):* Net recharge is the amount of water per unit area of soil that percolates to the aquifer. This is the principal vehicle that transports the contaminant to the ground water. Higher recharges increase the chances of the contaminant being transported to the ground-water table.
- *[A] Aquifer media:* The material of the aquifer determines the mobility of the contaminant traveling through it. An increase in travel time of the pollutant through the aquifer increases contaminant attenuation.
- *[S] Soil media:* Soil media is the uppermost portion of the unsaturated zone/vadose zone characterized by significant biologic activity. This, in addition to the aquifer media, determines the amount of water percolating to the ground-water surface. Soils with clays and silts have larger water holding capacity and thus increase the travel time of the contaminant through the root zone.
- *[T] Topography (slope):* The higher the slope, the lower the pollution potential due to higher runoff and erosion rates, which include pollutants that infiltrate the soil.
- *[I] Impact of vadose zone:* The unsaturated zone above the water table is referred to as the vadose zone. The texture of the vadose zone determines the travel time of the contaminant. Authors of this model suggest using the layer that most restricts water flow.
- *[C] Conductivity (hydraulic):* Hydraulic conductivity of the soil media determines the amount of water percolating through the aquifer to the ground water. For highly permeable soils, the travel time of the pollutant is decreased within the aquifer.

The major assumptions outlined in DRASTIC are:

- The contaminant is introduced at the surface.
- The contaminant reaches ground water by precipitation.
- The contaminant has the mobility of water.
- The area of the study site is more than 100 acres.

DRASTIC evaluates pollution potential based on the seven hydrogeologic settings listed above. Each factor is assigned a weight based on its relative significance in affecting the pollution potential. Each factor is also assigned a rating for different ranges of the values. Typical ratings range from 1 to 10, and weights range from 1 to 5. The DRASTIC Index, a measure of pollution potential, is computed by summation of the products of rating and weights of each factor as follows:

DRASTIC Index =
DrDw + RrRw + ArAw + SrSw + TrTw + IrIw + CrCw

where:

Dr = Ratings for the depth to water table
Dw = Weights for the depth to water table
Rr = Ratings for different ranges of aquifer recharge
Rw = Weights for the aquifer recharge
Ar = Ratings for the aquifer media
Aw = Weights for the aquifer media
Sr = Ratings for soil media
Sw = Weights for soil media
Tr = Ratings for topography (slope)
Tw = Weights for topography
Ir = Ratings for the vadose zone

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Iw = Weights for the vadose zone
Cr = Ratings for different rates of hydraulic conductivity
Cw = Weights for hydraulic conductivity

DRASTIC assigns two different weights depending upon the type of contaminant. Pesticides are given different weights than general contaminants. In assigning the weights, DRASTIC considers the different properties of pesticides as they travel through the vadose zone and root zone of the soil media.

The higher the DRASTIC Index, the greater the relative pollution potential. The DRASTIC Index is divided into four categories: low, moderate, high, and very high. The sites with high and very high categories are more vulnerable to contaminations and hence should be reviewed by the site specialist. These weights are relative, however. Low pollution potential does not necessarily indicate that a site is free from ground-water contamination. It indicates only that the site is less susceptible to contamination than sites with high or very high DRASTIC ratings.

SEEPAGE

The SEEPAGE model is a combination of three models adapted to meet the Soil Conservation Service's (SCS's) need to assist field personnel (7, 8). SEEPAGE considers hydrogeologic settings and physical properties of the soil that affect ground-water vulnerability to pollution potential. SEEPAGE is also a numerical ranking model that considers contamination from both concentrated and dispersed sources.

The SEEPAGE model considers the following parameters:

- Soil slope
- Depth to water table
- Vadose zone material
- Aquifer material
- Soil depth
- Attenuation potential

The attenuation potential further considers the following factors:

- Texture of surface soil
- Texture of subsoil
- Surface layer pH
- Organic matter content of the surface
- Soil drainage class
- Soil permeability (least permeable layer)

Each factor is assigned a numerical weight ranging from 1 to 50 based on its relative significance, with the parameter that has the most significant effect on water quality assigned a weight of 50 and the least significant assigned a weight of 1. The weights are different for concentrated or site-specific sources, and dispersed or nonspecific sources.

Similar to DRASTIC, each factor can be divided into ranges and ratings, varying from 1 to 50. The ratings of the aquifer media and vadose zone are subjective and can be changed for a particular region. Once the scores of the six factors are obtained, they are summed to obtain the SIN. These values represent pollution potential, where a high SIN implies relatively more vulnerability of the ground-water system to contamination. The SIN values are arranged into four categories of pollution potential: low, moderate, high, and very high. A high or very high SIN category indicates that the site has significant constraints for ground-water quality management (7).

GIS

GIS has been widely used for natural resources management and planning, primarily during the past decade. A GIS can be combined with a ground-water quality model to identify and rank the areas vulnerable to pollution potential for different scenarios and land use practices. Many GIS software packages are available. GRASS is a raster-based public domain software developed by the U.S. Army Construction Engineers Research Laboratory (9). This software can assign different weights to, or reclass, the data layers and combine map layers, and is suitable for implementing the DRASTIC and SEEPAGE models. ARC/INFO is a GIS software developed by Environmental Systems Research Institute (ESRI) in Redlands, California. The GRID submodule of ARC/INFO facilitates the handling of raster data. Also, the capability to develop a menu-based GUI helps users easily implement the models. The GRID submodule also can reclass and manipulate the map layers suitable for conducting the analyses.

Methodology

Developing the Data Layers in GRASS and ARC/INFO

The STATSGO database from SCS comes at a scale of 1 250,000 and is distributed in different data formats. This study used the STATSGO database in the ARC/INFO format. The database is organized into map units that have up to 21 components. These map components have information assigned to layers of soil horizons. Each layer is attributed various soil properties such as pH or organic matter content (10). Each property is assigned a high and a low value for a map unit. The STATSGO map for Indiana is available in the vector format. This map was exported to GRASS as a vector coverage (11) and was converted into a raster coverage within the GRASS GIS environment. This was used as

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the base map for the DRASTIC and SEEPAGE analyses. The hydrogeologic parameters required for the models were identified from the corresponding INFO data tables and were exported into an ASCII file. Code was developed to generate a GRASS reclass file assigning the weighted values of the parameters to the corresponding map units in the base map. The STATSGO base map imported into GRASS was reclassed for each hydrogeologic setting (e.g., topography, pH) to create the data layers required for DRASTIC and SEEPAGE analyses.

The map layers of the hydrogeologic parameters in GRASS were then exported to ARC/INFO as raster coverages. A GRASS command was developed that allows the output ASCII file from GRASS to be imported into ARC/INFO directly without further modifications to the header in the ASCII file.

Developing a Graphic User Interface in ARC/INFO

The dynamic form-menu option (12) was used to develop a GUI for both DRASTIC and SEEPAGE analyses (see Figures 1 and 2). Because ratings for some parameters are subjective, the GUI provided an option to change the weights assigned to hydrogeologic settings. The coverages must already be assigned ratings before using the interface, however. The interface also allows users to reclassify the final vulnerability maps qualitatively (13) into four categories (low, moderate, high, and very high) after viewing the range of DRASTIC Index or SIN values.

Conducting the Analyses

The data layers were developed separately for the high and low values of the hydrogeologic settings. Once all the data layers were compiled, the corresponding ratings and weights were assigned and the analyses were conducted using the GUI. The data layers aquifer recharge, aquifer media, and vadose zone media were not available, so the analyses were conducted without these base maps. The SEEPAGE analysis was performed for concentrated/point sources of pollution. The final vulnerability indexes from the analyses were classified into four categories (low, medium, high, and very high) (see Table 1) to generate the final statewide vulnerability maps.

Table 1. Pollution Potential Categories Using SEEPAGE and DRASTIC Indexes

	Range of DRASTIC/SEEPAGE Index			
Analysis	**Low**	**Moderate**	**High**	**Very High**
SEEPAGE	1-24	25-48	49-70	> 70
DRASTIC	30-70	71-100	101-110	> 110

Validating the Accuracy of the Vulnerability Maps

The ground-water vulnerability maps produced by DRASTIC analysis were compared with those generated using the SEEPAGE model. The final statewide ground-water vulnerability maps from either approach were compared with the well-water quality data sampled from over 2,500 wells (see Figure 3), and the number of wells falling into each vulnerability category was tabulated.

Results and Discussion

Statewide analysis of ground-water vulnerability to pollution potential was conducted using the DRASTIC and SEEPAGE analyses at a scale of 1:250,000 in the raster format. The analyses were conducted for both the high and low values of the hydrogeologic settings, and the final vulnerability maps were prepared for the state of Indiana (see Figures 4 and 5). The vulnerability maps from both approaches were compared in the GRASS environment.

In both analyses, the low values of hydrogeologic settings resulted in more areas being classified as high and very high categories, compared with the high values of hydrogeologic settings in a map unit. The DRASTIC analysis placed more areas in the very high vulnerability category, compared with the SEEPAGE analysis, which categorized the same areas as high vulnerability. The nitrate-nitrogen concentrations observed in the wells were compared with the final vulnerability maps, and the number of wells falling into each of the four vulnerability categories (low, moderate, high, and very high) were summarized (see Tables 2 and 3).

Approximately 80 percent of the wells with concentrations less than 5 parts per million are classified under the moderate vulnerability category in SEEPAGE analysis. Overall, the results from the analyses did not correlate satisfactorily with the observed well-water quality data. Unavailability of the data layers aquifer media, aquifer recharge, and vadose zone media may have caused these results. The well-water quality data of nitrate-nitrogen contaminations was considered only for testing map accuracy, whereas the analyses do not account for the type of contaminant, its severity, and its volume in the generation of vulnerability maps. Other limitations of these approaches, including that the factors influencing aquifer contamination (e.g., direction of water flow, land use, population at risk, point sources of pollution) are not considered in the ground-water vulnerability evaluation, might also have led to the observed results.

The DRASTIC and SEEPAGE analyses can be improved by incorporating data layers such as land use and nitrate loadings in computing the DRASTIC and SEEPAGE Indexes. The STATSGO database can be used for

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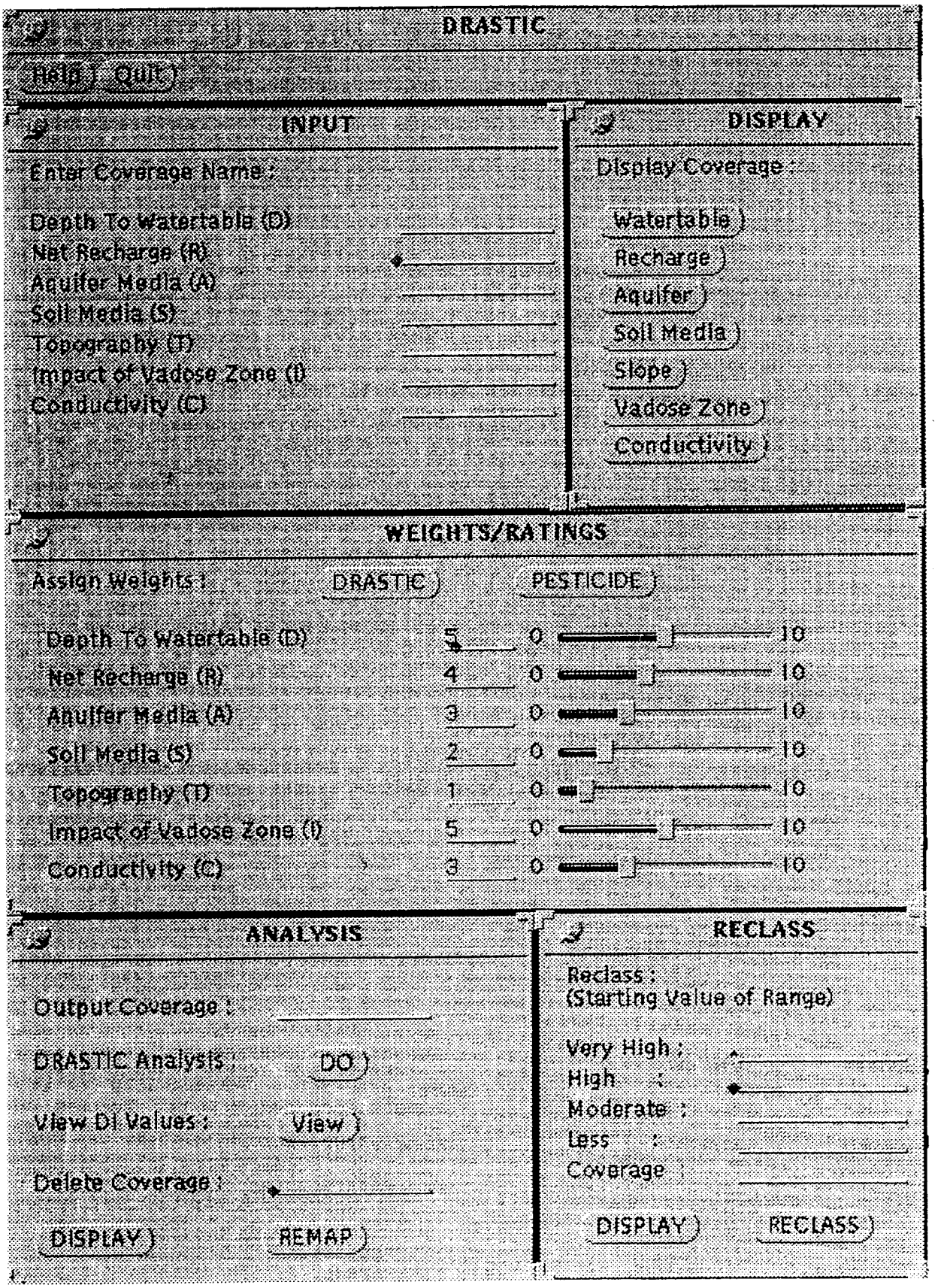

Figure 1. GUI for DRASTIC analysis.

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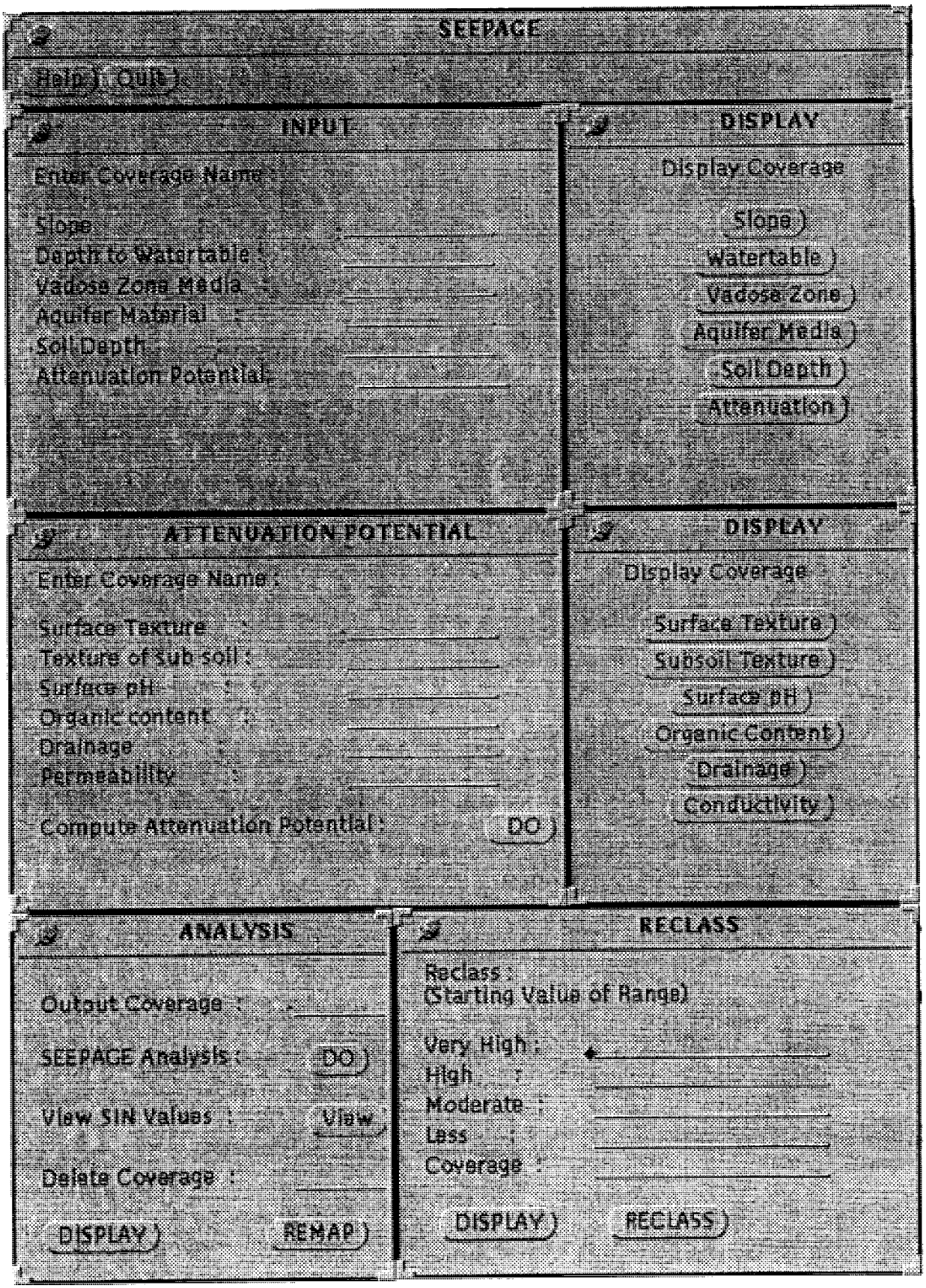

Figure 2. GUI for SEEPAGE analysis.

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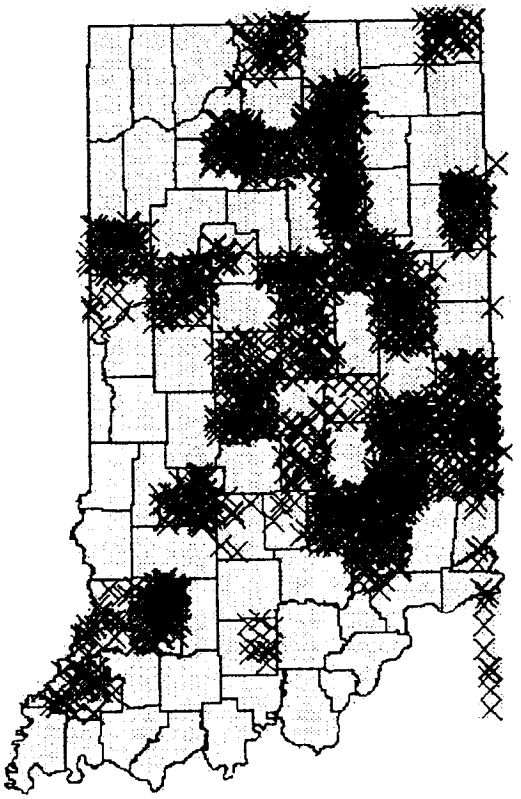

Figure 3. Sampling sites (wells) for water quality data.

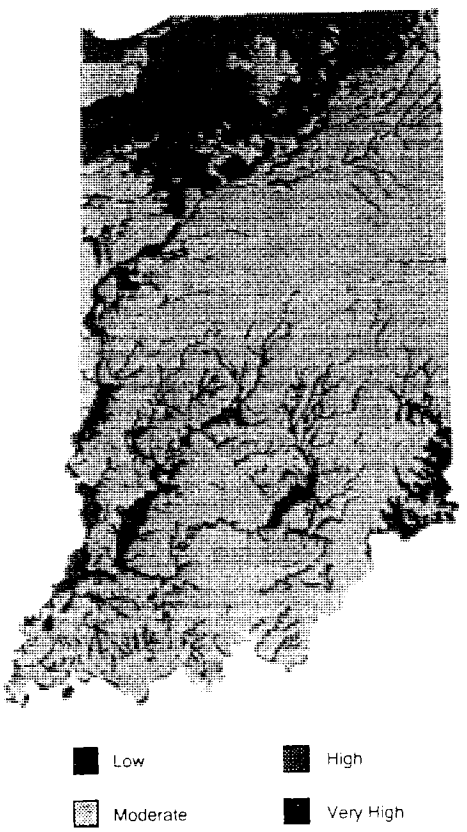

Figure 4. Ground-water vulnerability map using DRASTIC analysis.

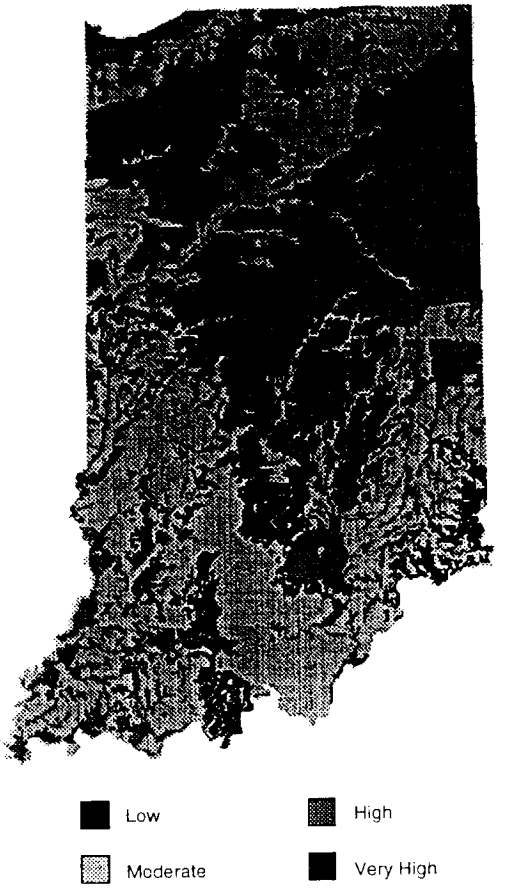

Figure 5. Ground-water vulnerability map using SEEPAGE analysis.

Table 2. Comparison of SEEPAGE Results With Observed Nitrate-Nitrogen Concentrations in Wells

	Nitrate-Nitrogen Levels		
Category	0-5	5-10	> 10
Low	7	2	8
Moderate	1,322	76	384
High	249	11	1
Very high	194	2	64

Table 3. Comparison of DRASTIC Results With Observed Nitrate-Nitrogen Concentrations in Wells

	Nitrate-Nitrogen Levels		
Category	0-5	5-10	> 10
Low	11	2	9
Moderate	541	41	290
High	720	39	213
Very high	500	9	65

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developing most of the data layers required for the analyses. GIS is a useful tool for integrating ground-water quality models and facilitates testing the models for different scenarios. The GUI helps users easily conduct analyses and facilitates changing the weights for subjective hydrogeologic settings. DRASTIC and SEEPAGE approaches show great potential as screening tools for policy decision-making in ground-water management.

Summary

Ground-water pollution from agricultural management systems is of wide concern. Few models address ground-water vulnerability on a regional scale. The DRASTIC and SEEPAGE models are numerical ranking models that consider various hydrogeologic settings affecting the contamination of a region. The data required for these models are commonly available data, and the STATSGO database at 1:250,000-scale was used in this study. These models were integrated in the GIS environment of GRASS and ARC/INFO in the raster format. A menu-based GUI was developed in ARC/INFO for conducting the analyses. The vulnerability maps generated from DRASTIC and SEEPAGE analyses were compared. The statewide vulnerability maps also were compared with the well-water quality data to validate the accuracy of the models.

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Verification of Contaminant Flow Estimation With GIS and Aerial Photography

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Abstract

Estimation of contaminant movement in ground water requires interpolation of data from sampling wells that represent a very small sample of aquifer volume. Spatial statistics and kriging provide the best unbiased estimator of interpolated concentrations. Hurricane Hugo provided an opportunity to compare these estimators with actual forest mortality caused by saltwater inundation associated with the tidal surge. During the 9- to 15- month period after the hurricane, salt from the tidal surge moved within the shallow water table aquifer, causing widespread tree mortality on Hobcaw Forest in eastern Georgetown County, South Carolina. A small watershed (12 acres) was instrumented with 24 multilevel sampling wells. Piezometric potential and samples for salt concentration were collected for 12 months (months 18 to 30 after the tidal surge). These data produced three-dimensional estimations of flow directions and two-dimensional maps of chloride concentration. These maps led to the identification of important heterogeneities in the water table aquifer. Apparently, the infiltrated salt water moved to the bottom of the aquifer (15 feet) and emerged, killing the forest, where aquifer heterogeneity resulted in upward movements of ground water.

Georgetown County implemented a geographic information system (GIS) for tax mapping in 1988 and prepared 1:400-scale orthophotographs of the entire county with true ground accuracy of less than 5 feet. Color infrared aerial photographs were taken from a Cessna 150 platform annually after the hurricane. ERDAS GIS software and the accurate photo base allowed removal of scale irregularities and distortion that resulted from using a small aircraft. Scanned images, using a 10-square-foot pixel, were compared with kriged chloride concentration maps, also using a 10-foot cell size. Grid cells with estimated chloride concentration of more than 500 milligrams per liter also exhibited low reflectance in the infrared-enhanced color band, indicating tree mortality. Here, a small number of sampling wells accurately predicted ground-water movement of a contaminant (NaCl), and GIS and remote sensing verified this movement.

Introduction

Estimating contaminant flow in ground water is difficult because we cannot "see" the aquifer. We know that aquifers comprise sediments that vary from place to place, that changes in hydraulic conductivity determine the rate of water movement, and that the spatial variability of the aquifer sediment determines the hydraulic conductivity. Our inability to accurately represent spatial variability of the aquifer limits our ability to predict ground-water flow and, thereby, contaminant transport.

A large variety of prediction models are available (1-3), and stochastic methods of estimating spatial heterogeneity have been developed (4, 5) and tested (6, 7). On well-characterized field sites, these techniques can produce predictions of tracer movements that accurately predict experimental plumes in terms of mass behavior. Even at these research sites, the spatial distribution of hydraulic conductivity is not known well enough to predict behavior at any particular point.

Ground-water measurements generally derive from wells that are single points. To understand movement of an entire plume, these single point samples must be extended to represent areas. The geostatistical approach allows quantitative estimation of the spatial variation of point estimates (8). Kriging is a technique that uses spatial covariance to estimate values at points where no measurement exists (9). It produces the best linear unbiased estimator of nonmeasured points (8).

Following Hurricane Hugo, these techniques were used to study saltwater movement in the water table aquifer in forested stands of eastern Georgetown County, South Carolina. Clemson University received a grant from the U.S. Forest Service to examine forest mortality and regeneration success within the forest zone covered by salt water during the tidal surge. In this study, we used a small sample of ground water to estimate the direction

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and concentration of salt moving in the aquifer. Geographic information systems (GIS) proved to be a useful tool to verify conclusions based on the small sample size. Onsite sampling, aerial photography, vector and raster GIS, and spatial statistics were combined into one analysis system. The system estimated and verified directions of salt movement within the aquifer. GIS and remote sensing of forest mortality produced an independent indicator of salt movement that could be compared with the geostatistical technique.

Problem Statement

The main goal of the research project was to evaluate problems for forest regeneration in areas covered by salt water during the hurricane. In many of these areas, the mature trees died during the summer following the hurricane. These areas have very low elevation, little relief, and abundant rainfall, causing the water table to remain near the soil surface. The hypothesis was that salt movement within the aquifer killed the mature trees and could limit regeneration success. We divided the problem into three tasks: to determine if salt concentrations in the aquifer were high in areas where mature trees died, to determine pathways of salt movement within the aquifer that could explain high salt concentrations, and to predict regeneration success from the pattern of salt movement.

GIS contributed both to testing the initial hypothesis and to extending predictions to areas not initially studied. GIS has been used primarily to store and display spatial data in a way that preserves and presents the spatial relationships as well as the data. For this project, we collected two dissimilar data types. To determine salt movement, we measured salt concentrations and piezometric pressures in a series of wells. GIS had to represent the well data and the domain of the kriging procedures in a coordinate system compatible with the mortality data. We determined forest mortality data from infrared-enhanced color aerial photography. GIS also had to allow separation of the infrared signature of the photography, transform the signature into data that were comparable with the well data, and ensure that the coordinate systems of the well data and the mortality data represented the same true ground positions.

To use the GIS ability for this project, we needed to choose several GIS parameters. In this case, we estimated ground-water chloride concentration using kriging, which produced data on a grid comparable with mortality interpreted from photographs. A raster GIS representation could be compared with individual grid chloride values. Each grid cell was 10 square feet so that each cell would be within a single tree crown.

Methods

Study Location

The study was located on 12 acres of a small watershed located on the eastern side of Hobcaw Forest, an experimental forest managed by Clemson University, Department of Forest Resources. Hobcaw Forest is located on the end of a peninsula between the Winyah Bay and the Atlantic Ocean in eastern Georgetown County, South Carolina. The study watershed is located immediately west of the salt marsh and barrier island separating the forest from the Atlantic Ocean and is in Pleistocene-aged beach sediment. Watershed divides were created by former low dune lines, and the stream is within a small depression between these former dune lines. Divides are from 7 to 8 feet above sea level and the stream from 4 to 6 feet above sea level.

The study watershed is 50 miles northeast of Charleston, South Carolina, where the eye of Hurricane Hugo struck the U.S. coastline. Along this portion of the South Carolina coast, the tidal surge was approximately 10 feet above mean sea level (10), covering the entire watershed. After the hurricane, shallow auger holes contained water with sodium concentrations of 4,000 milligrams per liter (11). The hurricane winds did little damage to the watershed forest, but 25 percent of the large oaks were windthrown (12). Beginning in the spring of 1990, however, many hardwoods and pines began dying. By the winter of 1990 and 1991, a large portion of the forest on the watershed had died. Tree mortality did not correspond with high salinity measured by the initial auger-hole method, suggesting movement in the water table aquifer.

Well Installation

The water table aquifer is about 20 feet thick, consisting of fine sand similar to the present beach, with thin beds of shells 10 feet beneath the stream. The bottom of the aquifer is a bed of clay up to 3 feet thick over a leaky artesian aquifer composed of shell and sand. Local rainfall recharges the water table aquifer. Recharge for the lower aquifer is provided by leakage from the water table aquifer beneath the center of the peninsula, about 2 miles west of the watershed, where land elevations are 15 to 25 feet above sea level. Piezometric potential in the lower aquifer is generally a few inches above the water table aquifer, making it only weakly artesian (13).

We installed 24 multilevel ground-water samplers (14) in the water table aquifer. Five samplers were located in regeneration measurement plots (15) placed within the stream. Two samplers, one at each edge of the hardwood wetland, formed a line perpendicular to the stream at the regeneration plot. Two more samplers were located along these lines near the watershed divides on each side of the watershed. The 24 samplers

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formed five transects across the stream (see Figure 1). Piezometric potential and ground-water chloride concentrations were measured from these samplers from March 31, 1991, through April 1, 1991. Williams (16) provides a complete description of samplers, sampling procedures, and laboratory analysis.

GIS Implementation and Measures

A GIS system for Hobcaw Forest management was developed in 1987 using Environmental Systems Research Institute's PC ARC/INFO software (17). The initial system consisted of forest stand boundaries digitized onto 1:100,000 digital line graphs (DLGs) purchased from the U.S. Geological Survey. These relatively crude maps were combined with stand records and used for management decisions that did not require exact locations of stand boundaries. Later, management of the endangered red-cockaded woodpecker's habitat required that mapped stand lines be closer to true ground locations than the original DLG data scale allowed. A program of ground surveys and aerial photography was conducted in the late 1980s to locate stand boundaries more accurately (18).

In 1988, Georgetown County began a program to convert county tax mapping to computer-based systems. The first step was to acquire survey grade orthophotography. Copies of 1:400-scale orthophotographs, with guaranteed ground accuracy of plus or minus 5 feet,

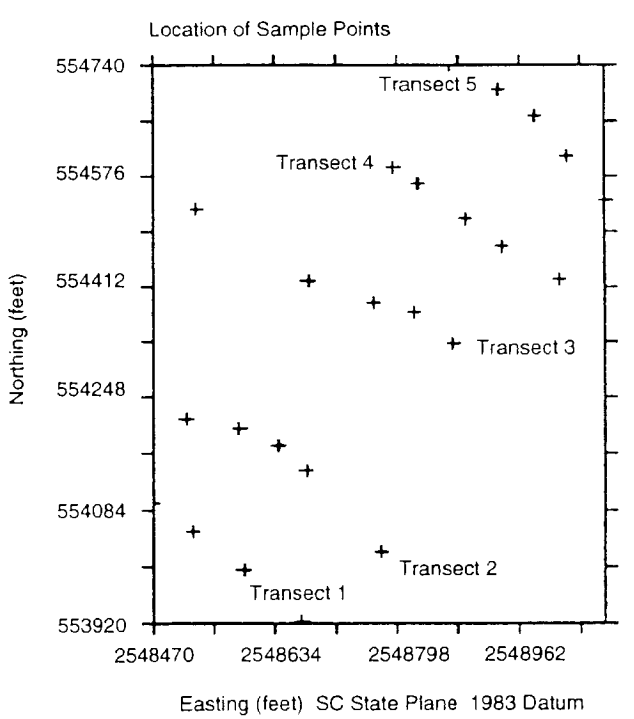

Figure 1. Position of sampling wells in rectangle defined for estimation of salinity movements.

became available in 1990. Roads and stand boundaries were digitized from these photographs into the PC ARC/INFO database. The new, accurate map was combined with stand records of previous coverages to create stand record coverages on a map that was true to the ground within 5 feet, plus or minus 0.3 percent.

In 1991, the GIS programs ERDAS VGA and LIVE LINK were obtained. ERDAS VGA programs allow image processing and raster GIS to be done on personal computers with VGA and some Super VGA monitor adapters. The LIVE LINK program allows display of both ARC/INFO and ERDAS images on the same monitor screen. Orthophotographs were scanned using 5-square-foot pixels and rectified with less than 1-pixel mean error, giving accurate ground locations of plus or minus 8 feet.

Ground surveys from a nearby benchmark provided accurate locations of the sampling wells. PC-TRAVERSE performed coordinate geometry from the survey notes, which was then plotted on the ARC/INFO forest stand database. Accuracy was checked by plotting the recognizable points on the survey with the scanned orthophotograph using the LIVE LINK software.

In February 1991, the Hobcaw Forest was photographed with infrared-enhanced color film. In this film, the red layer is sensitive to near infrared radiation that is strongly reflected by chlorophyll. Red colors in resulting prints indicate living vegetation. The color photography was not corrected for scale variation (from small fluctuation in aircraft altitude) or for distortion (caused by slight variations in the aircraft attitude). One photograph (1:1,320 scale) covering the study watershed was scanned into the ERDAS program. This image was rectified to the 1988 orthophotograph image using control points visible on both. A 10-square-foot pixel was used to sample individual tree crowns. The mean error of rectification was 1.5 pixels for a ground location, plus or minus 15 feet.

Ground-water chloride values at any one point varied over three orders of magnitude in all three dimensions and over two orders of magnitude with time. Annual averages of piezometric potential and chloride concentration, however, yielded interpretable results that were also statistically significant (16). Averaged values could then be combined with the surveyed sampler locations in the ARC/INFO system. The GIS also calculated the corner coordinates for a rectangle that would include all the sampler locations.

Geostatistical calculations were performed using the GS+ software. The data input to this program is an ASCII file of sample point locations in x,y coordinates and data values. The program allows calculation of semivariograms with various combinations of active and maximum lag distance and fitting of various model types to

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best fit the semivariogram (19). An active lag of 65 feet and a Gausian model produced the best fit:

$$\tau(h) = 0.001 + 1.337\ (1 - \exp\ [-h^2/20736]),\ r^2 = 0.616$$

where $\tau(h)$ is the semivariance at lag distance h.

This best fit model was then used in a block kriging (9) procedure. The procedure used eight nearest neighbors and calculated average values for 10- by 10-square-foot blocks. Values were calculated within the rectangle defined by the corner coordinates from the ARC/INFO procedure described above.

Finally, the rectangle defined around the sampler position formed the region of comparison between the rate of mortality, as sampled by infrared reflection, and estimated average chloride concentrations. The first comparison involved mapping reflection in the red band of the aerial photograph as a gray-scale map and comparing it with the contoured map of chloride concentration. The rectangle coordinates were used in the ERDAS software to create a subset of the scanned aerial photograph that included only the red band in the 5,145 pixels defined by the rectangle surrounding the samplers. In this subset, the infrared reflection was scaled as a gray-scale value between 0 and 255 for each 10- by 10-square-foot block defined in the concentration map. In addition to mapping, a regression of chloride concentration to gray-scale value was performed using the individual blocks.

Results

Ground Water

Ground-water chloride data reflected a consistent explanation of salt movement. Initial auger-hole data collected within a month of the hurricane indicated most salt was near the surface of the pine ridges, where, presumably, salt water had filled the aquifer to the soil surface during the hurricane. Data collected 30 months after the hurricane indicated the bulk of the salt had moved to the bottom of the aquifer under the pine ridges. Figures 2 and 3 represent the most significant results interpreted from the piezometric potential and chloride concentration measurements.

Figure 2 represents a cross section of the chloride concentrations and directions of ground-water flow in transect 4, the second most northern transect. The common information represented in this cross section was the west to east movement of ground water, representing the regional flow toward the forest edge. Also, there is an area of upwelling just east of the stream at the bottom of the aquifer, representing a leaky spot in the underlying clay layer. Upwelling causes the west to east streamlines to rise toward the surface along the western edge of the wetland. Chloride concentrations indicate large

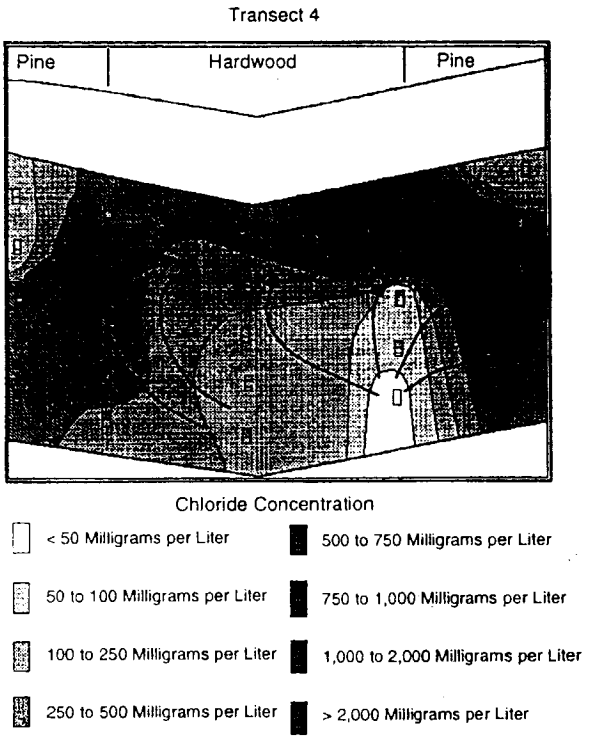

Figure 2. Cross section of aquifer at transect 4; gray scale represents chloride concentration, boxes at sampler positions represent 95-percent confidence limits of average chloride concentration in same gray scale, and arrows are perpendicular to contours of piezometric potential and represent two-dimensional vectors of streamlines.

reservoirs of salt beneath each of the pine ridges and small pockets of fresher water near the surface, probably the result of rain infiltration during the 30 months since the hurricane. The water upwelling from the artesian aquifer was consistently fresh. Flow passing through the concentrated zone beneath the western ridge was pushed to the surface beneath the stream, where evaporation caused chloride concentrations to average more than 1,000 milligrams per liter.

Figure 3 represents two plan views of the site at depths of 4 and 12 feet below the surface. The 12-foot plan view indicated that the east to west flow in transect 4 is only the east vector of a southeast flow. Other areas of upwelling exist beneath the stream. Chloride concentrations are highest beneath both ridges and lowest in the stream center. At the 4-foot depth, the east and southeast flows are also obvious. Also, as deeper flows from the western ridge are turned to the surface, high concentrations of chloride are present near the surface. High concentrations within the wetland result from water being carried to the surface due to upwelling within the wetland.

GIS Evaluations

The ground-water interpretations show a consistent explanation of salt movement. These interpretations are

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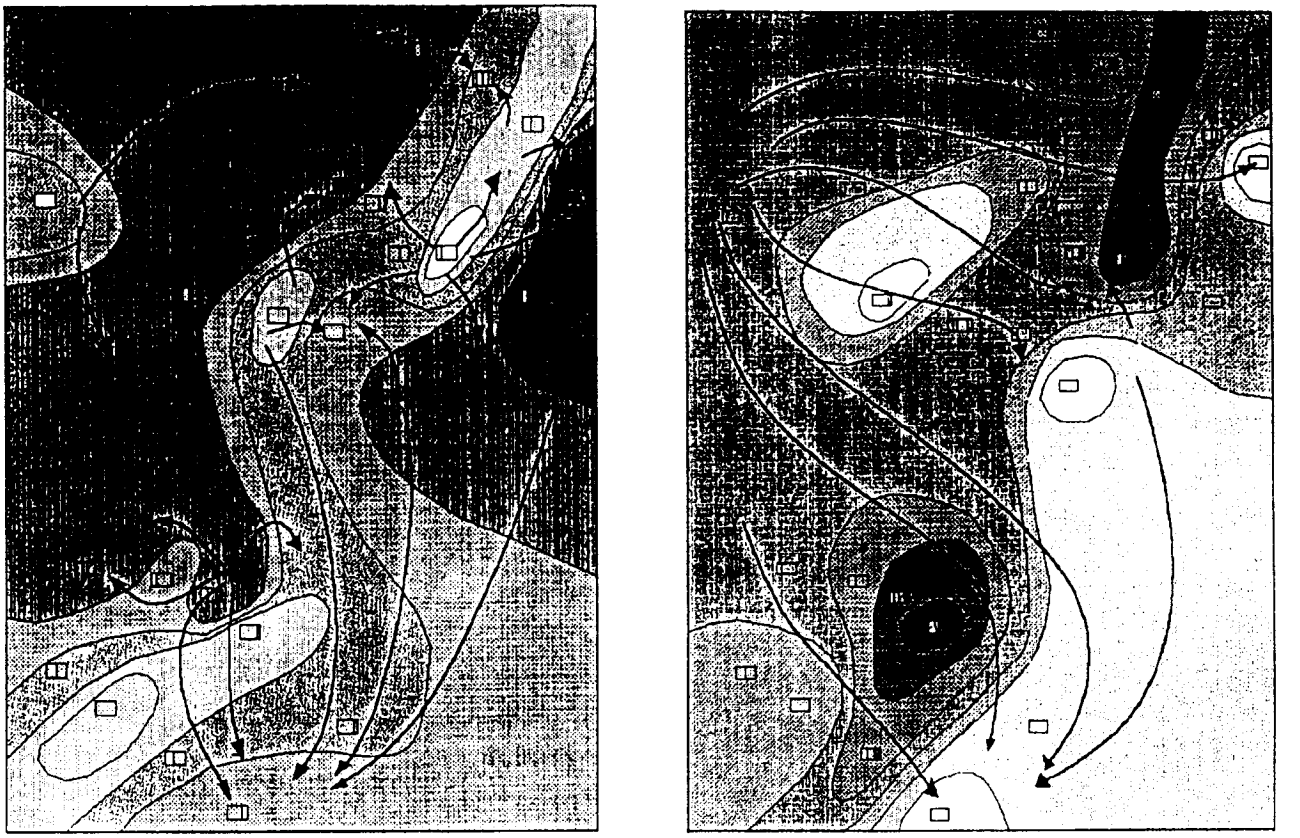

Plan View 12-Foot Depth

Plan View 4-Foot Depth

Figure 3. Plan views of aquifer at depths of 4 and 12 feet below the surface with same chloride scale and two-dimensional plan vectors of streamlines.

based on only 24 sample points. Interpolation was linear using the nearest neighbor. The samples removed from the aquifer represent only 0.000014 percent of the aquifer volume. Interpretation of a three-dimensional flow regimen from such small sampling does not produce great confidence in the validity of the interpretation.

Salt at the 4-foot depth is most likely to interact with tree roots, and concentrations at this depth were used for kriging. Kriged results, mapped in the same manner as the aerial photography, show general agreement of high mortality and predicted chloride concentrations over 500 milligrams per liter (see Figure 4). A regression of chloride concentration (chloride) to gray-scale value (G) for the individual points yielded a significant negative correlation. The regression line G = 169 - 0.12 (chloride) explained only 27 percent of the variation in gray-scale value, however.

Conclusions

GIS was successfully used to verify interpretations of ground-water flow and salt movements in a shallow water table aquifer. A variety of computer software combined to create a system of analysis that allowed integration of field sampling, aerial photography, vector and raster GIS, and spatial statistics. Using this system, we compared chloride movement, measured by subsurface samplers, with remotely sensed tree mortality caused by soil salinity. The overall pattern of mortality was predicted by a 500-milligram-per-liter chloride contour estimated by kriging averaged concentrations. Estimation of mortality on a single tree basis was less successful, with a regression of chloride to infrared reflection explaining only 27 percent of the variation in reflection. The regression did not fit values of high reflection well but did predict reflection values of 100 or below (regions of high mortality) for concentrations above 500 milligrams per liter.

The most important factor in the success of this project was the availability of large-scale orthophotography. Georgetown County's investment in accurate mapping allowed creation of a map base that made scale correction of less costly aerial photography possible. Without assurance that pixels on the aerial photograph corresponded to the same locations as the subsurface samplers, correlations would have been meaningless.

Another factor that contributed greatly to the research was the fact that most of the computer software exported or imported data from simple ASCII files. The standard (x coordinate, y coordinate, data value) format in ASCII allowed files to be manipulated with spreadsheets

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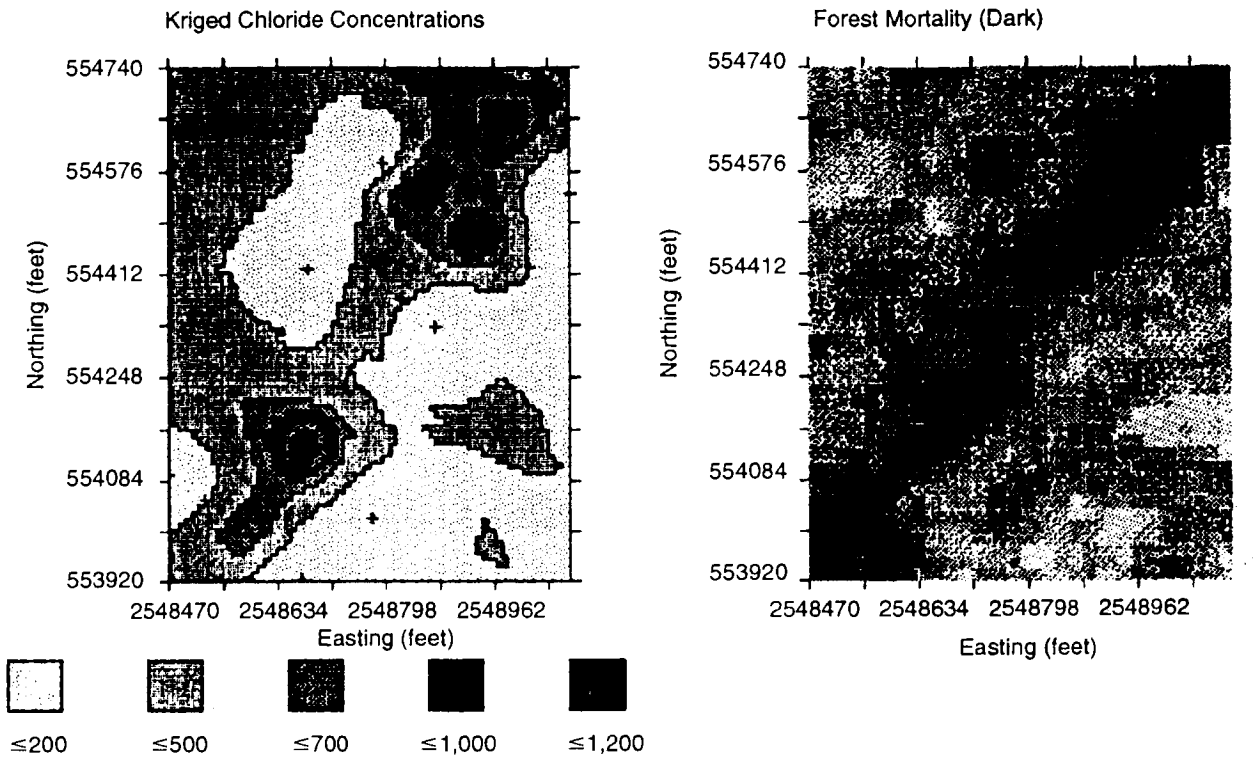

Figure 4. Plan view of chloride concentrations from krig analysis and gray scale of infrared reflection from aerial photograph. Lighter tones represent greater infrared reflection and less forest mortality.

or word processors. Creation of headers, positioning of columns, or changing order of rows or columns could be done for import into the next program. Although more difficult than point and click file transfers of the modern software, the simple standard format creates freedom to use the software in ways not anticipated by the software developers.

Finally, a clear problem statement aided in selecting the most applicable GIS techniques. GIS software allows several methods of data representation. In this example, we chose a raster representation with a cell the size of a tree crown. Criteria for choosing these parameters included physical dimensions of the phenomenon of interest, dimensions of GIS accuracy, and a desire for automated determination of values for individual comparisons. A careful review of the problem to be solved, data available, and capabilities of the GIS software are all necessary ingredients for a useful problem statement.

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Geology of Will and Southern Cook Counties, Illinois

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Introduction

The Silurian dolomite aquifer is the primary source of ground water in northeastern Illinois. It is overlain by glacially derived sands and gravels or tills. The sands and gravels within the glacial drift hydrologically interact with the fractured and creviced dolomite bedrock.

The purpose of this study was to define the extent of major glacial drift aquifers and their relationship to the shallow bedrock aquifer surface. The study succeeded in identifying two principal sand and gravel aquifers: an "upper" drift aquifer within the glacial tills and a "basal" drift aquifer overlying the bedrock. Bedrock topography, drift thickness, thickness of the Silurian dolomite, and thickness of major sand and gravel units were mapped to help define the geologic and hydrologic system and the interaction of the upper bedrock aquifer and the drift aquifers.

The data collected to create the various maps came from well records, engineering borings, oil and gas tests, and structure tests on file at the Illinois State Geological Survey (ISGS). Reviewing published reports, manuscripts, and unpublished reports on open file at the ISGS provided an overall perspective of the geology of the study area. Previously, no detailed studies of the hydrogeology of the entire area had been conducted. Incorporating water well and other data into a computer database greatly facilitated map construction. Preliminary maps were developed using Interactive Surface Modeling (ISM) software and a geographic information system (GIS).

Past regional geologic studies of the northeastern Illinois area that have encompassed this study area include Thwaites (1), Bretz (2), Bergstrom et al. (3), Bretz (4), Suter et al. (5), Hughes et al. (6), and Willman (7). Bogner (8) and Larsen (9) included interpretive maps of the surficial geology of the area as a part of planning studies for northeastern Illinois.

Map Construction

Creating the database used in the construction of the maps for this project entailed inputting information from well driller's logs into a PC-based computerized spreadsheet (Quattro Pro). Well logs were primarily from water wells and engineering borings. Data items input into the spreadsheet included:

- Well identification (ID) number
- Owner name
- Location of well
- Thickness of drift
- Depth to top and bottom of the bedrock
- Depth to top and bottom of each sand unit

The ground surface elevation of each well was interpolated from United States Geological Survey (USGS) 7.5-minute quadrangles. Elevations of the top of bedrock and top and bottom of sand bodies were calculated based on the interpolated elevations. Locations were verified wherever possible using plat books by matching either landowner names or the address location from the well log. After compilation, the data were converted to ASCII text and transferred into an ARC/INFO (Versions 5.0.1 and 6.0) database on a SUN SPARC workstation. ARC/INFO is a product of Environmental Systems Research Institute, Inc., of Redlands, California.

Of the more than 10,000 records reviewed for this project, over 5,100 were input into the database. Subsequently, numerous data quality checks ensured that duplicate well ID numbers were corrected, locations were corrected, thicknesses were checked so that the sand thickness data reported did not exceed drift thickness, and elevations were checked so that elevation of a sand body was not below the bedrock surface. After running the data quality checks and removing questionable data from the database, approximately 5,000 records remained.

ISM, a contouring package from Dynamic Graphics, Inc., of Alameda, California, helped to create two dimensional grid representations of:

- Surface topography
- Drift thickness

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- Bedrock topography
- Bedrock isopach
- Intermediate sand body isopach
- Basal sand isopach

ISM also allowed for the creation of contoured output of the grids. Grids are regularly spaced rectangular arrays of data points (nodes) that allow for efficient mathematical calculations and contouring. ISM uses a minimum tension gridding technique, allowing for the curvature (change in slope) of the surface to be spread throughout the surface rather than being concentrated at the input data points. The ISM program uses a biharmonic inverse cubic spline function (algorithm) to assign data values to grid nodes. This function assumes that for any grid node assignment, input data points farther away from the node being evaluated have less influence on that node's value than nearer data points. To determine each grid node value, ISM calculates an average value from the surrounding scattered input data (up to 15 input data points) and finds the standard deviation. ISM continues to refine the values of the grid nodes until the standard deviation is minimized (10).

Several grid spacings were reviewed to determine which would best represent the density of the data. The grids that ISM uses, as described above, determine the fineness to which the data control the resultant contours. Experimentation was necessary to determine a grid spacing that adequately represented the data. Too fine a grid spacing can exaggerate or overly weight individual points, causing the resultant contours to be overly jagged. With too large of a grid spacing, the contours can become overgeneralized and become much less data dependent because the calculated grids are overaveraged.

The two-dimensional grid of the land surface topography was based on surface topography lines and spot elevations digitized from USGS 7.5-minute quadrangles. The linework for each quadrangle was converted to ASCII files of data points. The ASCII files contained x and y coordinates and the elevation value of each data point. After inputting the ASCII files into ISM, a two-dimensional grid for each quadrangle was created. ISM also generated contour lines from each grid. Comparing plots of the generated lines with USGS 7.5-minute topographic maps allowed for the correction of errors and ensured that the grid elevation values were within 10 feet of the elevations shown on the USGS maps. An ISM two-dimensional grid of the entire area's surface topography was created by combining the grids. After creating a contoured surface of the grid, an ARC/INFO coverage of the output was produced. ARC/INFO was used to edit the coverage and produce the final map.

The two-dimensional grid of the bedrock surface topography was based on data from water well and engineering boring logs, ISGS field observations of outcrop locations, and previous ISGS mapping (9). An ASCII file of x and y coordinates and the elevation of each bedrock top was input into ISM. Subtracting the bedrock topography grid from the land surface grid produced a grid of the drift thickness. A contoured output of the grid was produced, and an ARC/INFO coverage of the output was created. Again, ARC/INFO was used to edit the coverage and produce the final map.

Creating the isopach maps entailed subtracting the top and bottom elevations of each unit to calculate the thickness of each unit. ASCII files of the x and y coordinates and the thickness values for each data point were input into ISM. ISM then created two-dimensional grids of each isopach. Contoured output of each grid was produced, which allowed for the creation of ARC/INFO coverages of the output. ARC/INFO was used to edit the coverages and produce the final maps.

Bedrock Geology of the Study Area

All the sedimentary bedrock units are of the Paleozoic Era. The Paleozoic bedrock comprises sequences of sandstones, dolomites, limestones, and shales. The stratigraphic column of Figure 1 illustrates the vertical succession of the bedrock. Major tectonic activity of the area includes the formation of the Kankakee Arch in Ordovician time (11) and faulting along the Sandwich Fault Zone. Faulting along the Sandwich Fault Zone (see Figure 2) may have occurred coincidentally with the formation of the Lasalle Anticlinorum in early Pennsylvanian time (12). No further faulting has been noted since deposition of glacial sediments. Bedrock units gently dip to the east (7). The majority of the area lies on the Niagara cuesta, a south and west facing scarp that comprises the resistant Silurian strata that have an eastward dip of roughly 15 feet per mile (13). The Silurian strata are absent west of the Kankakee River as well as in an area west of the Des Plaines River in west-central Will County (see Figure 2). This study relates to the hydrogeology of the Silurian strata and the drift materials, and details only the uppermost bedrock units. The report, however, does briefly summarize units below the Maquoketa Group using information from Hughes et al. (6) and Visockey et al. (14).

Precambrian Bedrock

Granites or granitic rock compose the Precambrian basement of northern Illinois. Few details about the nature of the basement rocks are known because few wells have completely penetrated the sedimentary bedrock of the region. The elevation of the top of the Precambrian basement probably stands at 4,000 feet below mean sea level in the study area.

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SYSTEM	SERIES	GROUP OR FORMATION	AQUIFER	LOG	THICKNESS (FT)	DESCRIPTION
QUATERNARY	PLEISTOCENE		Sands and Gravels		0 – 250	Unconsolidated glacial deposits-pebbly clay (till), silt, sand and gravel Alluvial silts and sands along streams
PENNSYLVANIAN	DES MOINESIAN	Spoon and Carbondale			0 – 110	Shale, sandstone, clay, limestone, and coal
SILURIAN	NIAGARAN	Racine	Silurian (Shallow dolomite aquifer system)	reef	0 – 350	Dolomite, very pure to argillaceous, silty, cherty; reefs in upper part
		Sugar Run				Dolomite, slightly argillaceous and silty
		Joliet				Dolomite, very pure to shaly and shale, dolomitic; white, light gray, green, pink, maroon
	ALEXANDRIAN	Kankakee			0 – 100	Dolomite, pure top 1' - 2', thin green shale partings, base glauconitic
		Elwood				Dolomite, slightly argillaceous, abundant layered white chert
		Wilhelmi				Dolomite, gray, argillaceous and becomes dolomitic shale at base
ORDOVICIAN	CINCINNATIAN	Maquoketa			80 – 250	Shale, red to maroon, oolites Shale, silty, dolomitic, greenish gray, weak (Upper unit) Dolomite and limestone, white, light gray, interbedded shale (Middle unit) Shale, dolomitic, brown, gray (Lower unit)
	CHAMPLAINIAN	Galena	Galena-Platteville (Cambrian-Ordovician aquifer system)		310 – 380	Dolomite, and/or limestone, cherty (Lower part) Dolomite, shale partings, speckled Dolomite and/or limestone, cherty, sandy at base
		Platteville				
		Glenwood	Glenwood-St. Peter		125 – 600	Sandstone, fine and coarse-grained; little dolomite; shale at top Sandstone, fine to medium-grained; locally cherty red shale at base
		St. Peter				
	CANADIAN	Shakopee New Richmond Oneota Gunter	Prairie du Chien		0 – 410	Dolomite, sandy, cherty (oolitic); sandstone Sandstone interbedded with dolomite Dolomite, white to pink, coarse-grained cherty (oolitic) Sandstone, medium-grained, slightly dolomitic
CAMBRIAN	CROIXAN	Eminence	Eminence Potosi		0 – 280	Dolomite, light colored, sandy, thin sandstones
		Potosi				Dolomite, fine-grained gray to brown, drusy quartz
		Franconia	Franconia		110 – 160	Dolomite, sandstone and shale, glauconitic, green to red, micaceous
		Ironton	Ironton-Galesville		135 – 235	Sandstone, fine to coarse-grained, well sorted; upper part dolomitic
		Galesville				
		Eau Claire			390 – 570	Shale and siltstone, dolomitic, glauconitic; sandstone, dolomitic, glauconitic
		Elmhurst Member	Elmhurst-Mt. Simon aquifer system		2200	Sandstone, coarse-grained, white, red in lower half, lenses of shale and siltstone, red, micaceous
		Mt. Simon				
PRE-CAMBRIAN						Granitic rocks

Figure 1. Generalized stratigraphic column of rock units and aquifers in northern Illinois (prepared by M.L. Sargent, ISGS).

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Cambrian

The Elmhurst-Mt. Simon Sandstone comprises the oldest sedimentary units in Illinois and consists of medium-grained sandstones. It has a total thickness of approximately 2,500 feet. The upper part of this unit has acted as an aquifer in the Chicago region in the past; ground-water mining of the aquifer (a nonreplenished lowering of the static water level), however, has led to a discontinuation of its use for that purpose. The Eau Claire Formation, the Basal Sandstone Confining Unit (14), consists of dolomitic shale and siltstone with thin beds of sandstone. It has a thickness of 300 feet to 400 feet and separates the Elmhurst-Mt. Simon aquifer from the Ironton-Galesville Sandstones. The Ironton-Galesville Sandstones have a thickness of 150 feet to 250 feet and serve as a source of ground water in northern Illinois (6). The Galesville Sandstone is fine-grained, while the Ironton Sandstone is coarser grained and contains more dolomite. The Knox Megagroup, the Middle Confining Unit (14), comprises all the bedrock units between the Ironton-Galesville Sandstones and the Ancell Group. It includes the:

- Cambrian Franconia Formation
- Potosi Dolomite
- Eminence Formation
- Jordan Sandstone
- Ordovician Prairie du Chien Group

The Knox Megagroup is primarily dolomitic in composition, though it contains thin sandstones. Its thickness ranges from 400 feet in the northern portion of the study area to about 700 feet in the southernmost tip of Will County. The sandstones tend to be somewhat discontinuous and, where present, offer a localized source of ground water. The group as a whole acts as a confining unit between the Ironton Sandstone and the Ancell Group.

Ordovician

The Ancell Group, which contains the St. Peter Sandstone and Glenwood Sandstone, has a thickness of roughly 200 feet throughout the study area except in north-central Will County where it is over 400 feet. The thickness of the Ancell Group varies considerably in northern Illinois because it rests on an erosion surface. The Ancell Group is the shallowest aquifer present in this area below the Silurian dolomite aquifer. The elevations of the top of the Ancell Group range from just over sea level in the northwest corner of Will County to 500 feet below mean sea level in the southwestern corner. The Galena and Platteville Groups provide a sequence of carbonate rocks that are primarily dolomitic in composition. The Platteville Group conformably overlies the Ancell Group. The two units have a combined thickness of 350 feet throughout this part of the state. The Galena and Platteville Groups, combined with the overlying Maquoketa Shale Group, act as an aquitard between the Ancell aquifer and the Silurian dolomite aquifer.

Maquoketa Shale Group

The study area has three subaerially exposed bedrock units. The oldest of these that this report details are Ordovician-aged strata comprising the Cincinnatian Series Maquoketa Shale Group. The thickness of the Maquoketa Group ranges from 260 feet in eastern Will County to 120 feet in the northwestern corner of Will County and is unconformably overlain by Silurian strata (15). The Maquoketa Group comprises four formations:

- Scales Shale
- Fort Atkinson Limestone
- Brainard Shale
- Neda Formation

The Scales Shale forms the lowermost unit and consists of gray to brown dolomitic shale. Thin layers with phosphatic nodules and pyritic fossils occur near the top and base of the unit. The Scales Shale may attain a thickness of up to 120 feet in this region (15). The Fort Atkinson Limestone, a coarse-grained crinoidal limestone to fine-grained dolomite, may range up to 60 feet thick (15). The Brainard Shale comprises greenish gray dolomitic shale and has a thickness of generally less than 100 feet (16). The Neda Formation, the youngest formation in the Maquoketa Group, is relatively thin with a thickness of usually less than 10 feet. In some places, it may attain a maximum thickness of 15 feet. The Neda is exposed along the Kankakee River, and the Silurian-aged Kankakee Formation typically overlies it. The Neda Formation consists mostly of red and green shale with interbedded goethite and hematite oolite beds (7, 16).

Silurian System

Silurian-aged rocks consist almost solely of dolomites and dolomitic limestones. The Silurian is divided into the Alexandrian and Niagaran Series. The Alexandrian Series is about 25 feet thick and is represented by the Kankakee, Elwood, and Wilhelmi Formations. These formations are a fine- to medium-grained, white, gray to pinkish gray dolomite. The Kankakee Formation is exposed along the Kankakee River in southern Will County (17).

The Niagaran Series comprises much of the bedrock surface of this area and includes three formations. The Joliet Formation has a lower member of dolomite with interbedded red and green shale, and two upper members with an increasing purity of dolomite toward the top of the formation (7). The Sugar Run Formation, formerly termed the Waukesha Formation (17), is an argillaceous, fine-grained, medium- to thick-bedded,

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brownish gray dolomite (7). The Racine Formation is the thickest unit in the Niagaran Series, attaining a thickness of as much as 300 feet (17). The Racine Formation contains large reefs that are as high as 100 feet and consist of vugular gray dolomite. The inter-reef rock consists of dense, cherty gray dolomite. The Racine Formation is exposed in the bluffs along the Des Plaines River from Joliet to Blue Island, Illinois (17).

Figure 2 is an isopach of the Silurian dolomite indicating the thickness of the unit in the study area and the boundary of the Silurian rocks. The Silurian dolomite aquifer has a maximum thickness of just over 500 feet in the southeast corner of Will County and becomes thicker to the east and south. It rapidly increases in thickness from its margin along the western border of Will County, where it has eroded. The contact between the Silurian dolomite and the underlying Maquoketa Shale Group has relatively little relief. Thus, the major differences in thickness of the unit result from erosion of the bedrock surface. Joints and fracture patterns within the upper bedrock have a dominantly northwest-southeast and northeast-southwest orientation (18).

Pennsylvanian System

Pennsylvanian-aged bedrock is found in the southeastern portion of Will County west of the Kankakee River with an outcropping at the confluence of the Des Plaines and Kankakee Rivers. The lowermost unit, the Spoon Formation, is very thin and consists of clay beds with scattered occurrences of coal formed in channel-like depressions (19). The Spoon Formation overlies the Maquoketa Shale Group. The overlying Carbondale Formation may attain a thickness of over 100 feet in the southwestern corner of Will County. The Carbondale Formation consists of shale with thin limestone beds. The lowermost unit, the Colchester (Number 2) Coal Member, outcrops in this area and attains a thickness of up to 3 feet. It has been extensively mined along the Will-Grundy-Kankakee County border where large areas of strip-mined land are evident. Most of the available coal has been mined out, and numerous gob piles exist

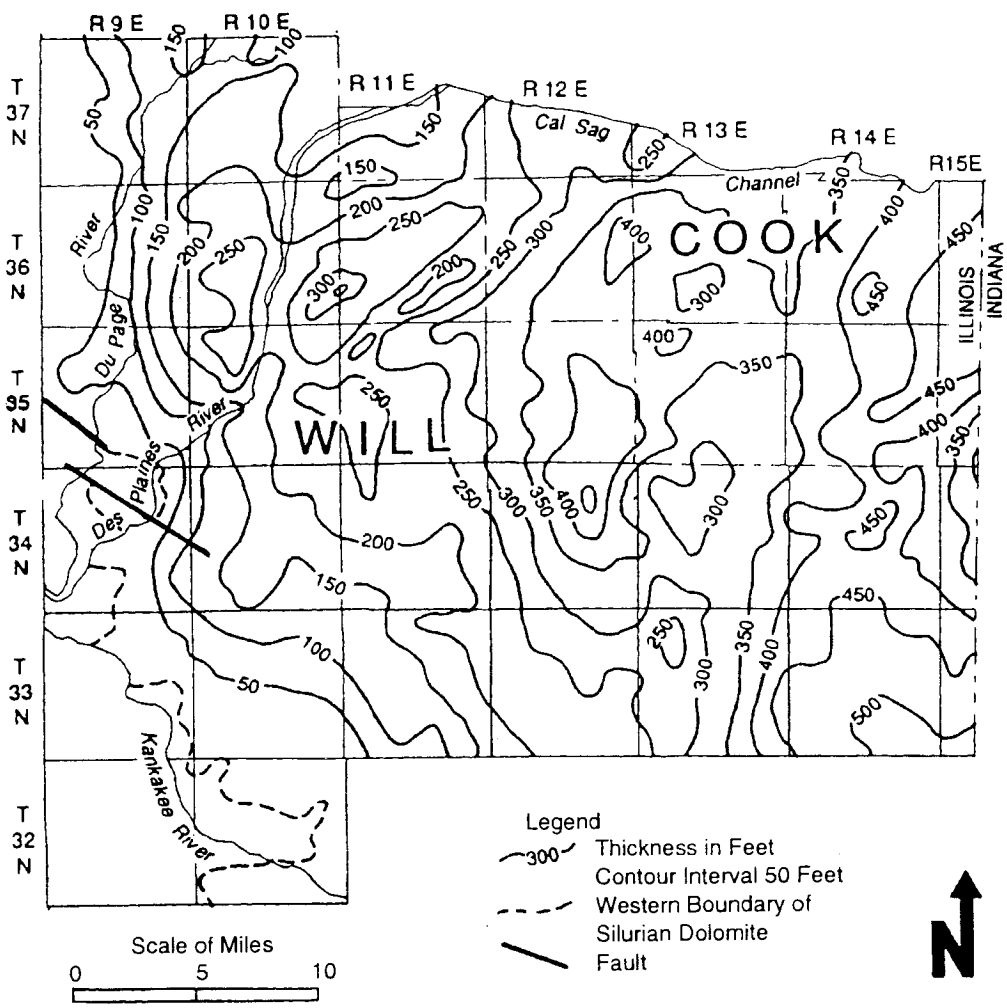

Figure 2. Thickness of the Silurian dolomite.

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in the area of Braidwood. The Francis Creek Shale Member, which overlies the Number 2 coal, constitutes the remainder of Pennsylvanian units in the study area. The Francis Creek Shale is gray with numerous flattened concretions that contain the Mazon Creek flora of Pennsylvanian-aged fossils (19). Weathering of the mine slag materials may have exposed fossiliferous concretions in the gob piles (7).

Bedrock Topography

The highest bedrock elevations are in east-central Will County where the bedrock rises to over 700 feet above mean sea level (see Figure 3). Bedrock uplands occur as a broken curved ridge from the southeast to the northwest with bedrock elevations consistently rising over 650 feet above mean sea level. The bedrock surface slopes from the bedrock upland high westward to the Des Plaines River. It also has a regional downward slope to the south into Kankakee County, Illinois, to the northeast into the Lake Michigan basin, and to the east into Indiana. West of the Des Plaines River, the bedrock surface rises to over 650 feet above mean sea level in northeastern Kendall County. Elsewhere, the surface has relatively low relief.

The dominant features of the bedrock surface are the river valleys. The Des Plaines River valley is better expressed than the Kankakee River valley. This is true, in part, because it is older and acted as a drainageway for glacial meltwater where it may have become entrenched in the present valley. The Kankakee River valley may be less expressed partly because of the amount of scouring that occurred over a large area during the Kankakee flood event such that the river is not entrenched in most places. Also, smoothing of the study's contour maps has generalized some of the detail.

The buried Hadley Bedrock valley, described initially by Horberg and Emery (20), probably existed prior to glaciation and concurrently with the preglacial Des Plaines River. The valley may have acted as a drainageway for glacial meltwaters until the time that glacial debris buried it. Glacial scouring was originally believed to have formed the valley, but evidence presented by McConnel indicated a fluvial origin of the valley (21). Also, the base of the Hadley valley does not overhang or lie much below the Des Plaines valley but rather joins it at a smooth juncture.

The bedrock surface contains a number of sinkholes or closed depressions that are expressions of karst

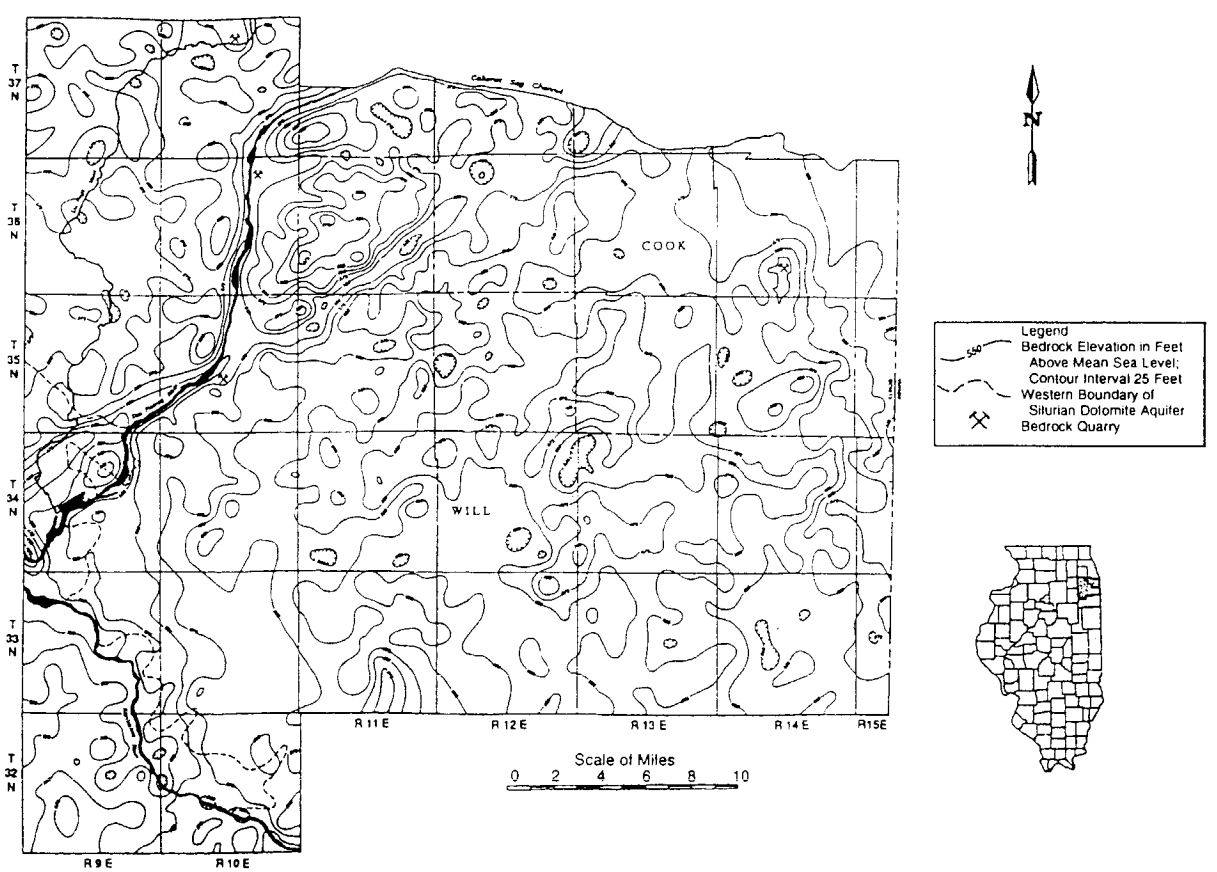

Figure 3. Topography of the bedrock in the study area.

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development that formed prior to continental glaciation. Karst, a terrain developed on limestone or dolomite by solution or dissolving of the rock, is characterized by closed depressions and cavity development along joints and fractures. Fischer (22) first noted karst features in the Joliet area where early Pennsylvanian sediments of shale and clay filled cavities in the upper bedrock. Buschbach and Heim (23) indicated closed depressions in the Silurian dolomite surface in their bedrock topography map for the Chicago region. They speculated these depressions were expressions of karst development. McConnel (21) demonstrated the existence of sinkholes in the area of the buried Hadley Bedrock valley northeast of Joliet by using seismic refraction survey data.

Glacial Geology

The sediments overlying the bedrock comprise tills, sands and gravels, lacustrine deposits from glacial lakes, and surficial eolian deposits of loess and sand. The unconsolidated deposits are over 150 feet thick along the crest of the Valparaiso Morainic System. Figure 4, adapted from Willman (24), indicates the principal moraines. In the area where the Hadley Bedrock valley is present, the deposits attain a thickness of over 175 feet. Bedrock is mainly exposed along the Des Plaines River valley and its tributaries. It is also exposed in isolated areas in southeastern Cook County. The drift thickness map (see Figure 5) indicates the distribution of the earth materials overlying the bedrock and the locations of bedrock outcrops. The bedrock outcrop information for this map was derived from Piskin (25) and Berg and Kempton (26).

Erosion of the glacial sediments was a major factor in controlling the drift thickness of the area. Succeeding glaciers scraped off previously deposited sediments, but glacial meltwaters, which came from the east and north along the river channels, caused much of the erosion. Both the Kankakee River and Des Plaines River acted as meltwater channels as the glaciers melted. The Du Page River acted as a minor drainageway and was most active during large-scale flooding events. The thickness of the drift varies in the area also because of the topographic control that the bedrock on the overlying sediments exercises. The crest of the Valparaiso moraines coincides with the topographic high in the bedrock

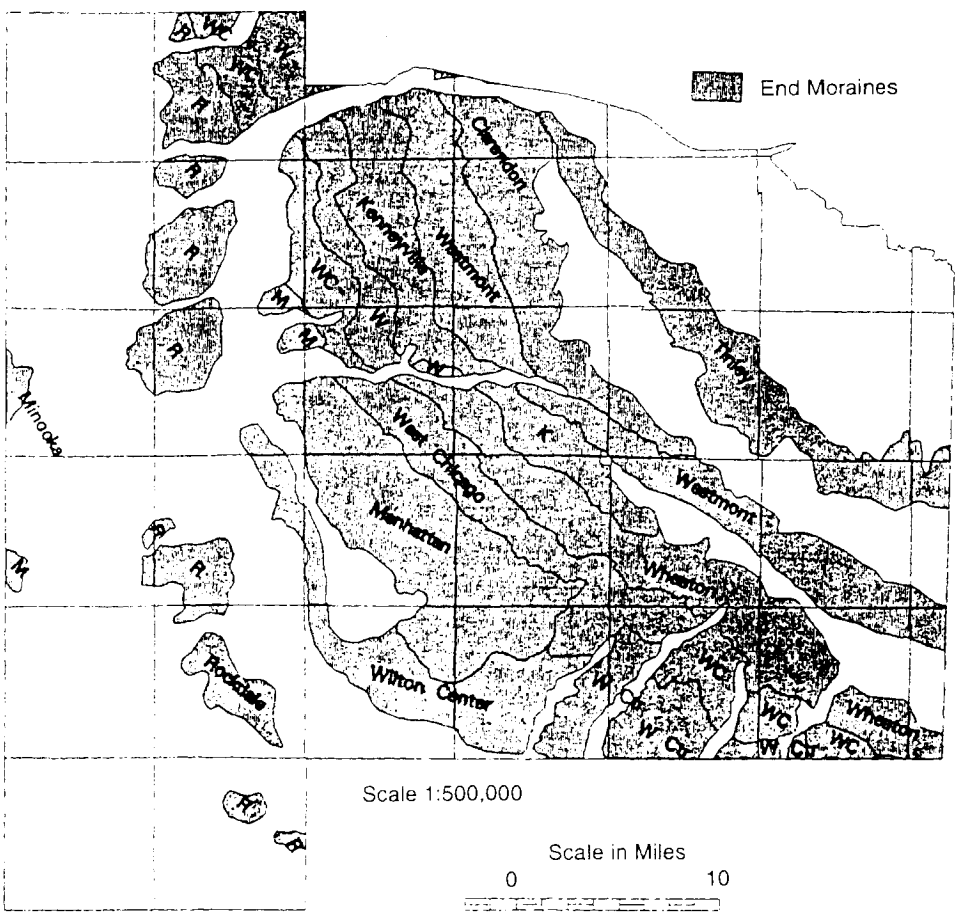

Figure 4. End moraines (late Wisconsin) in Will and southern Cook Counties, Illinois.

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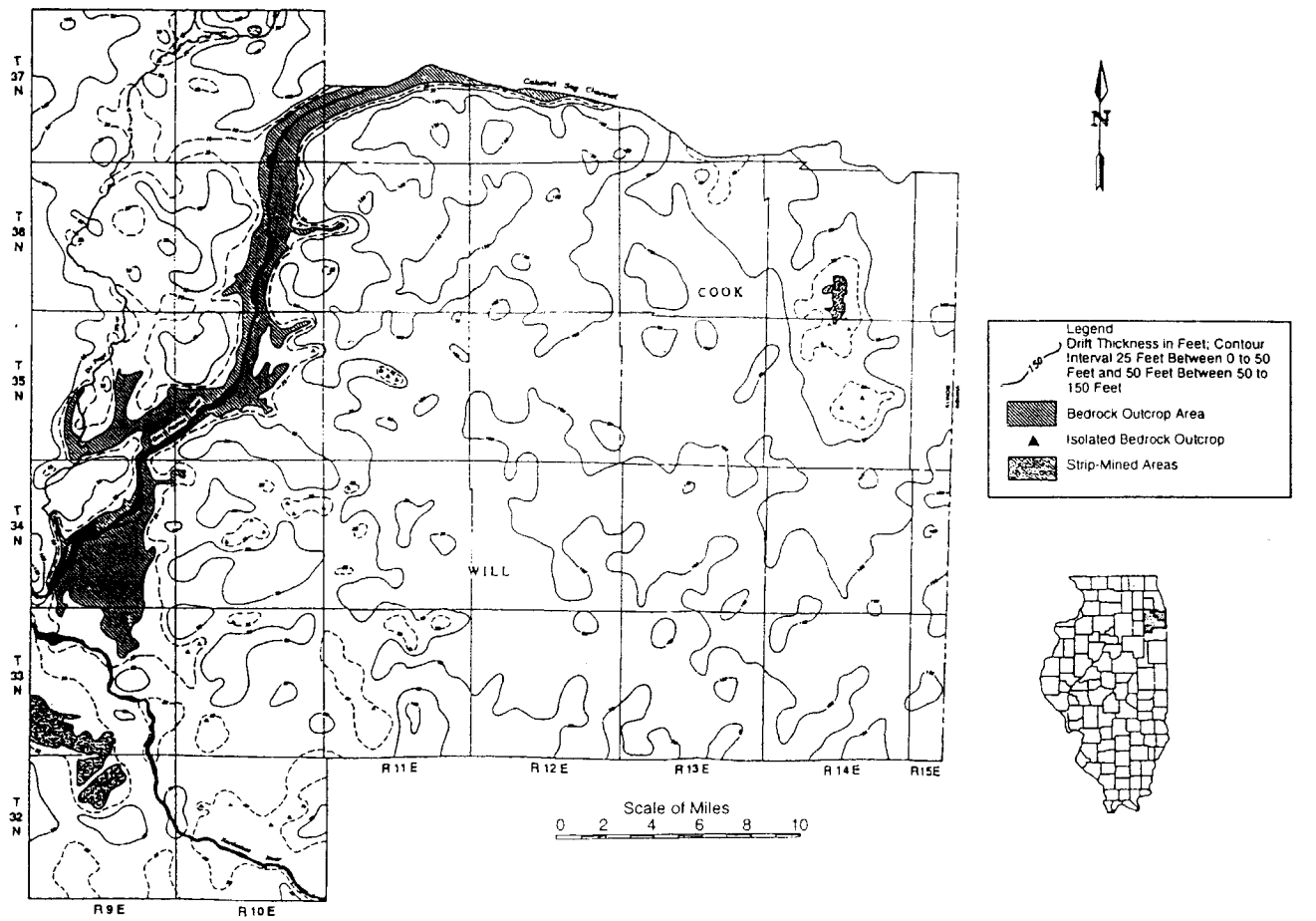

Figure 5. Thickness of the glacial drift in the study area and bedrock outcrop information (25, 26).

surface. The cross sections in Figure 6 also show this. The bedrock high may have caused late Woodfordian glaciers to stall repeatedly in the same area, causing moraines to build atop one another sequentially (27).

Descriptions by well drillers note few variations in the character of the unconsolidated sediments; therefore, we did not attempt to correlate these deposits. The drift materials present in the study area are late Wisconsinan or younger. Though this region experienced glaciating repeatedly prior to the Wisconsinan glaciation, no Illinoisan or pre-Illinoisan deposits have been identified (28). The drift units divide into three main units (29):

- The Lemont Drift
- The Yorkville Drift
- The Wadsworth Drift

The three drift units are all part of the Wedron Formation of Wisconsinan age. The Lemont Drift has a dolomitic character because the source material for the diamicton was glacially eroded Silurian dolomite. The Lemont Drift is the oldest of the three units and is found only underlying the Wadsworth Drift. The Yorkville Drift is the only drift unit present west of the Valparaiso Morainic System boundary within the study area. It overlies the bedrock surface wherever the basal sand unit is not present. The Wadsworth Drift comprises silty and clayey diamictons and is the youngest of the drifts (29). It overlies the Lemont Drift and the upper sand unit. In the cross sections (see Figure 6), where the upper sand unit is present, it roughly indicates the boundary between the Lemont and Wadsworth Drifts. The gradation between the different drift units at the Valparaiso System boundary is not well defined. The Wadsworth Drift appears to grade into the Yorkville Drift because they are very similar in composition near the boundary (9).

The large Kankakee flood left extensive deposits of sand and gravel and lacustrine sediments along the Kankakee River and Des Plaines River. The flood occurred as glacial meltwaters built up behind a constriction at the Marseilles Morainic System to the west (30). Large glacial lakes, which developed during the flood, subsequently emptied into the Illinois River valley after a breach in the moraines developed. The force of the flood waters eroded the glacial deposits along the river valleys, flattened the surface of the drift, and, in places,

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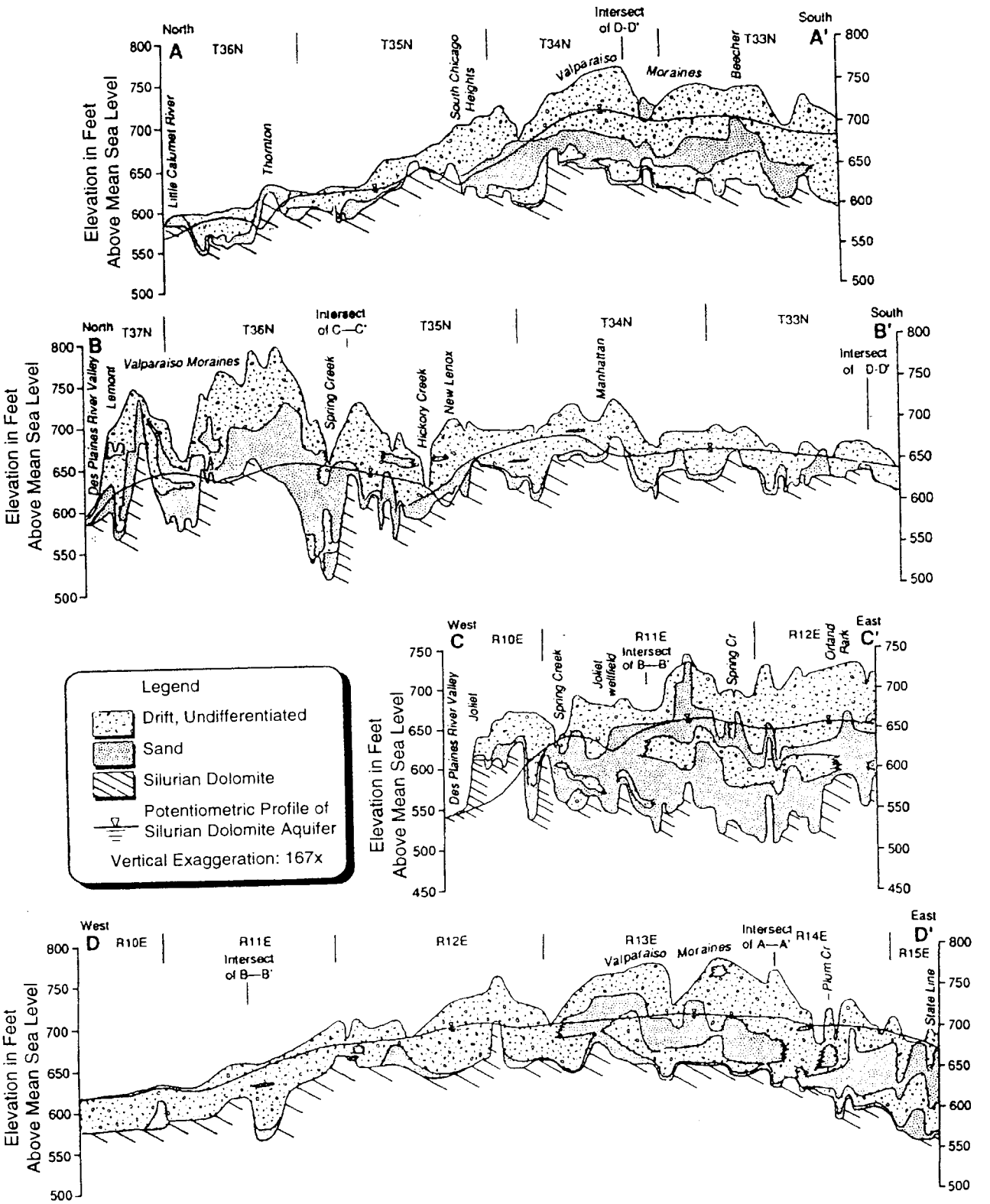

Figure 6. Geologic cross sections of the glacial drift and potentiometric profile of the Silurian dolomite aquifer.

exposed the underlying bedrock. The flood event formed thin, dispersed lake plain deposits of silt, clay, and sand in southwestern Will County. Some lacustrine deposits lie between morainic ridges in southern Cook County where small glacial lakes developed as the Valparaiso Moraines were being deposited (8).

Figure 7 shows the locations of some of the surficial materials. Sands and gravels were also deposited along tributary creeks and in abandoned channels that once connected the Du Page River and Des Plaines River north of their present juncture. Wind has reworked the surficial sand deposits forming low dunes along the Kankakee River in southern Will County. Masters (31) classified the sand and gravel deposits of the area by their origin, indicating that most of the deposits present in the valley of the Des Plaines River formed as well-sorted valley train deposits. In the Kankakee River valley, the sands and gravels were primarily deposited as riverine sediments during the Kankakee flood event.

Sand and Gravel Isopachs

The sand and gravel isopach maps (see Figures 8 and 9) indicate the variations in thickness of the upper and basal sand and gravel units. The most extensive deposits of both exist throughout the area overlain by the Valparaiso Morainic System. This may be associated with bedrock control on the formation of the moraines and associated deposits referred to earlier.

The thickest deposits lie in the buried Hadley Bedrock valley where thicknesses of both units can exceed over 100 feet. The upper sand unit may be found in the glacial drift within a wide range of elevations. For mapping purposes, we defined the upper sand unit as a sand unit greater than 1 foot thick that occurs between two fine-grained layers. The basal sand unit includes all coarse-grained materials that overlie the bedrock surface. Most of the basal sands present west of the Des Plaines River were formed as valley train deposits along the river

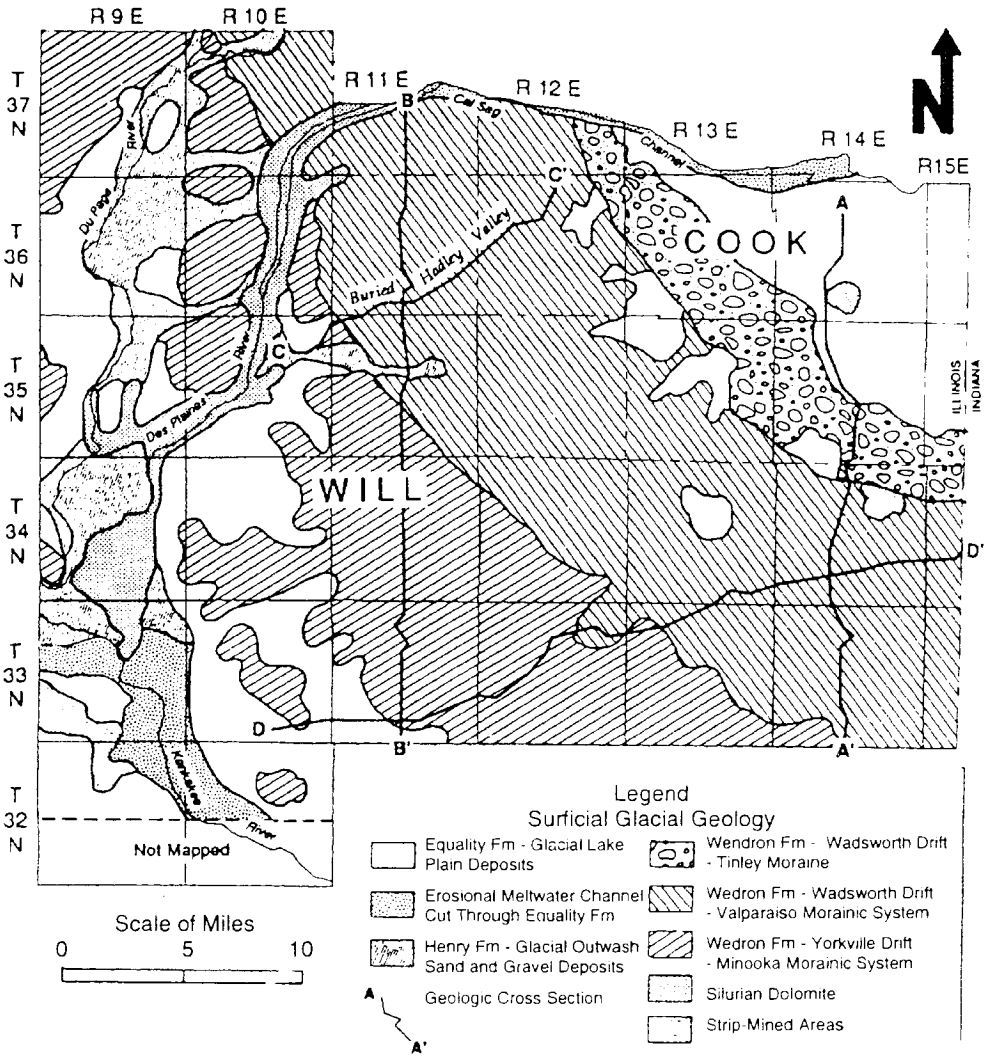

Figure 7. Surficial glacial geology (23).

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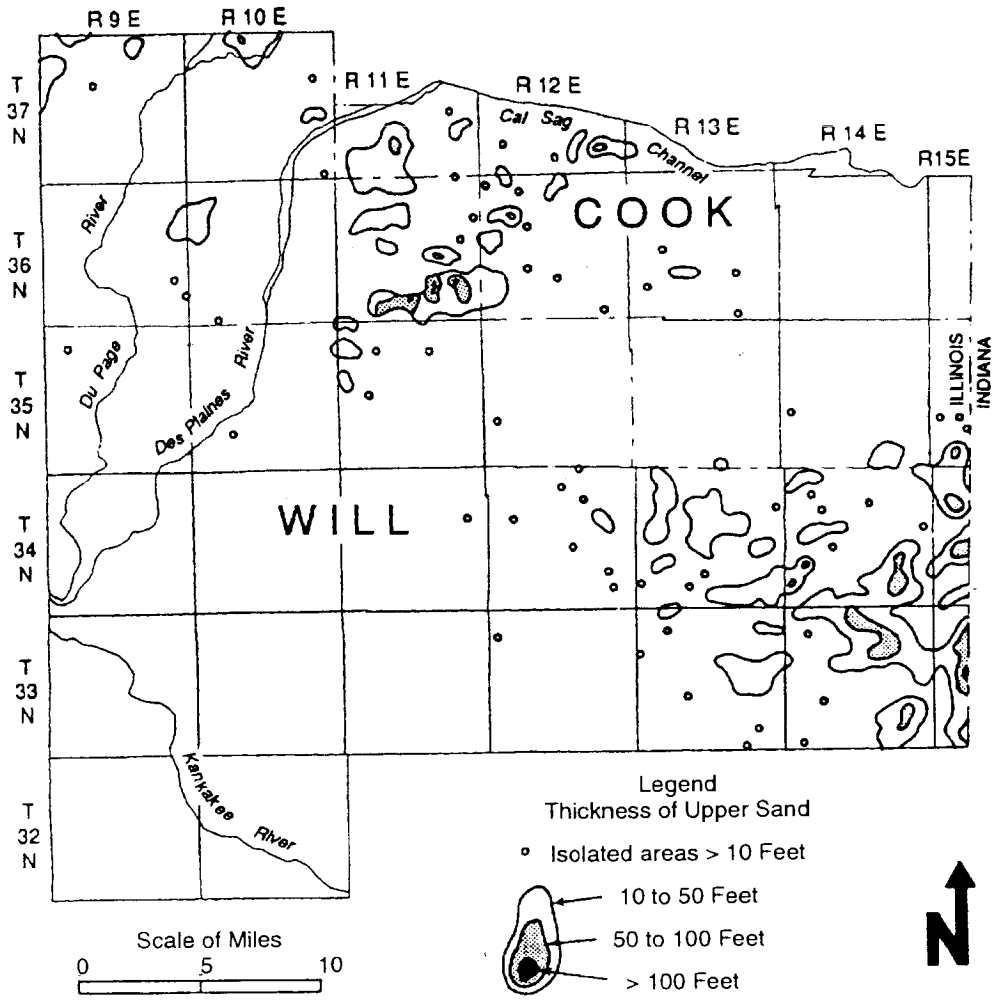

Figure 8. Thickness of the upper sand unit.

channels as the glaciers melted back. The origin of the extensive deposits underlying the Valparaiso Moraines is not clear. They may have been formed during early Wisconsinan glacial events as outwash plain deposits or they may have been deposited subglacially. The cross sections (see Figure 6) can reveal the variability and complexity of the sand and gravel layers as they occur within the drift. The sand and gravel deposits very seldomly act as aquifers in this region because almost all wells are completed in the Silurian dolomite aquifer. Clearly, Figures 8 and 9 indicate that some groundwater resource potential may exist within these deposits.

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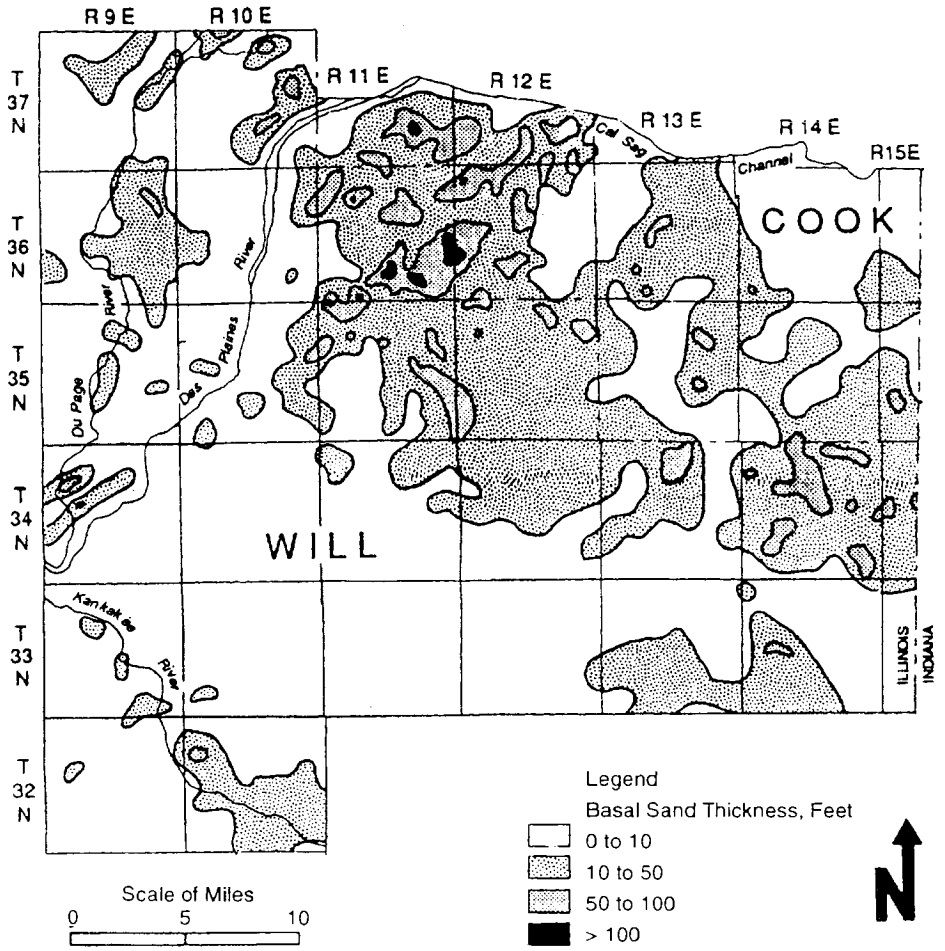

Figure 9. Thickness of the basal sand unit.

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Watershed Applications

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The Watershed Assessment Project: Tools for Regional Problem Area Identification

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The St. Johns River Water Management District of Florida recently completed a major water resources planning effort. As part of this planning effort, the St. Johns River Water Management District created a geographic information systems (GIS) project called the Watershed Assessment, which included a nonpoint source pollution load model. This paper introduces the planning project and the Watershed Assessment, and describes how the results of the model are being used to guide water management activities in northeast Florida.

Background

The St. Johns River Water Management District (District), one of five water management districts in Florida, covers 12,600 square miles (see Figure 1). The St. Johns River starts at the southern end of the District and flows north; it enters the Atlantic Ocean east of the city of Jacksonville. The cities of Orlando, Daytona Beach, and Jacksonville are partially or entirely within the District boundaries. Ad valorem taxes provide primary funding for the District.

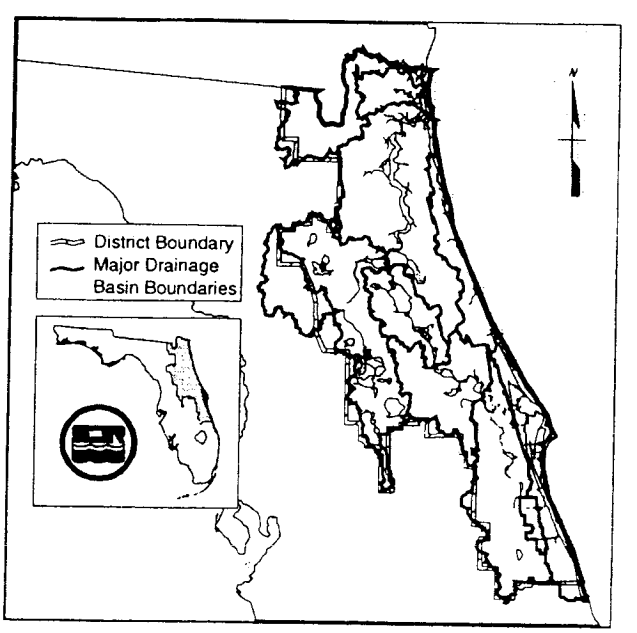

Figure 1. St. Johns River Water Management District, Florida.

The District boundaries are somewhat irregularly shaped because Florida water management districts are organized on hydrologic, not political, boundaries, which greatly improves the District's ability to manage the resources. On the north, the District shares the St. Mary's River with the state of Georgia and on the south, shares the Indian River Lagoon with another water management district. Most of the water bodies the District manages, however, have drainage basins that are entirely contained within the District's boundaries.

Water management districts in Florida have amassed extensive GIS libraries, which they share with local and statewide agencies. These libraries include basic data layers such as detailed land use, soils, and drainage basins. Districts also coordinate data collection and management to ensure data compatibility.

District Water Management Plan

All activities and programs of the water management districts are related to one or more of the following responsibilities: water supply, flood protection, water quality management, and natural systems management.

Each water management district recently completed a district water management plan (Plan). The main purpose of these Plans is to provide long-range guidance for the resolution of water management issues. The Florida Department of Environmental Protection will use these five Plans as the basis for a state water management plan. Each water management district used the same format, which comprised the following components:

- Resource assessment: What are the problems and issues related to each of the four responsibilities listed above?

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- Options evaluation: What options are available for addressing the problems?
- Water management policies: What existing District policies influence the decisions that must be made?
- Implementation strategy: What is the best plan for addressing the problems?

The Watershed Assessment Project

The District created the Watershed Assessment project as part of is resource assessment. This GIS project examines the entire District to identify problems related to flood protection, ecosystems protection, and surface water quality.

The flood protection component is the only part of the Watershed Assessment that is not complete. It will involve simple overlays of floodplain boundaries with existing and future land use. Floodplain boundaries are defined as Federal Emergency Management Agency (FEMA) flood insurance rate map 100-year flood hazard areas. In many areas, these designations are not very accurate, yet we decided to proceed with their use because they are the best available information for many parts of the District. In areas where little hydrologic information is available and where the District has not conducted any related studies, the FEMA data are a helpful starting point. This echoes a theme of the Watershed Assessment project: the assessment is primarily intended to fill in gaps where we have not performed previous resource assessments, not to supplant existing information.

The ecosystems protection component of the Watershed Assessment is based heavily on a project identifying priority habitat in Florida, conducted by the Florida Game and Fresh Water Fish Commission (1). It is similar to gap analyses that the U.S. Fish and Wildlife Service currently is conducting in many parts of the country. For the Watershed Assessment, we modified the data somewhat and examined ways to protect the habitat in cooperation with local agencies.

The surface water quality component of the Watershed Assessment has two main parts. The first uses water quality data from stations that have been spatially referenced so that we can map them and combine the information with other information, such as the second part of the water quality component. This second part is a nonpoint source pollution load model, which is discussed in more detail below.

The Pollution Load Screening Model

The nonpoint source pollution load model is the Pollution Load Screening Model (PLSM), a commonly used screening tool in Florida. It is an empirical model that estimates annual loads to surface waters from stormwater runoff. Our goal in designing this model was to identify pollution load "problem areas" for examination in the Plan.

In these types of models, annual pollutant loads are a function of runoff volume and mean pollutant concentrations commonly found in runoff. Runoff volume varies with soil and land use, while pollutant concentrations vary with land use. For the PLSM, pollutant concentrations were derived from studies conducted solely in Florida. A report describing the model in detail is available (2).

Usually, this kind of model combines GIS with a spreadsheet: the GIS supplies important spatial information that is input into a spreadsheet where the actual calculations are made. The PLSM is different, however, because we programmed it entirely within GIS. The District's GIS software is ARC/INFO, and the model employs an ARC/INFO module called GRID, which uses cell-based processing and has analytical capabilities (3). All the model calculations are done in the GIS software, resulting in a more flexible model with useful display capabilities.

Model input consists of grids, or data layers, with a relatively small cell size (less than 1/2 acre). We chose this cell size based on the minimum mapping unit of the most detailed input data layer (land use) and the need to retain the major road features. The model has four input grids: land use, soils, rainfall, and watershed boundaries. For any given cell, the model first calculates potential annual runoff based on the land use, soil, and rainfall in that cell. It then calculates annual loads by applying land-use-dependent pollutant concentrations to the runoff.

For this model:

- Land use is from 1:24,000-scale aerial photography flown in 1988 and 1989. The model incorporates 13 land use categories.
- Soils are the Soil Conservation Service (SCS) SSURGO database, which corresponds to the county soil surveys. The PLSM uses the hydrologic group designation of each soil type.
- Rainfall was taken from a network of long-term rainfall stations located throughout the District.
- Watersheds were delineated by the United States Geological Survey (USGS) on 1:24,000-scale, 7.5-minute maps and digitized.

Model output consists of a runoff grid and six pollutant load grids. We calculated loads for total phosphorus, total nitrogen, suspended solids, biochemical oxygen demand, lead, and zinc. We chose these pollutants because reliable data were available and because they characterize a broad range of nonpoint pollution-generating land uses, from urban to agricultural. The model

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calculates runoff and loads for any point in space, allowing the user to see the spatial distribution of loads. An example of a total phosphorus load grid for one subbasin in the Jacksonville, Florida vicinity is shown in Figure 2.

The grids themselves provide a detailed view of model output. Model results can also be summarized by watershed, using the watershed boundary grid, and the information can be examined from a basinwide perspective.

We have applied PLSM results in other useful ways at the District. For example, District staff felt that previous sediment sampling sites were not appropriately located, so the District water quality network manager used model results to locate new sampling sites, focusing on problem areas as well as areas where we expect to see little or no nonpoint impact.

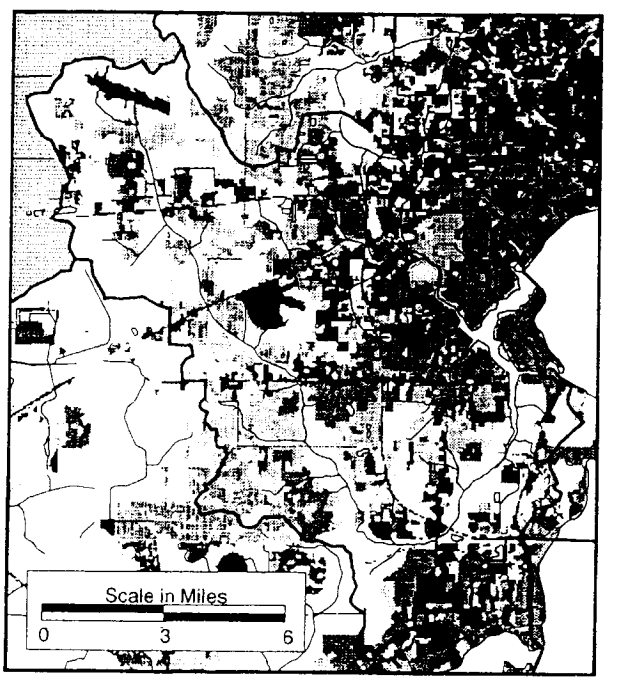

Figure 2. Distribution of total phosphorus loads, Ortega River subbasin (darker areas represent higher loads).

Application of Model Results in the Plan

Because the goal of the model was to identify potential stormwater runoff problem areas, we needed to simplify, or categorize, the model results for use in the Plan. We calculated the per acre watershed load for each pollutant and defined "potential stormwater runoff problem areas" as those individual watersheds with the highest loads for all pollutants. Problem areas for one major basin in the District, the lower St. Johns River basin, are depicted in Figure 3.

We also ran the model with future land use data obtained from county comprehensive plans. Because the

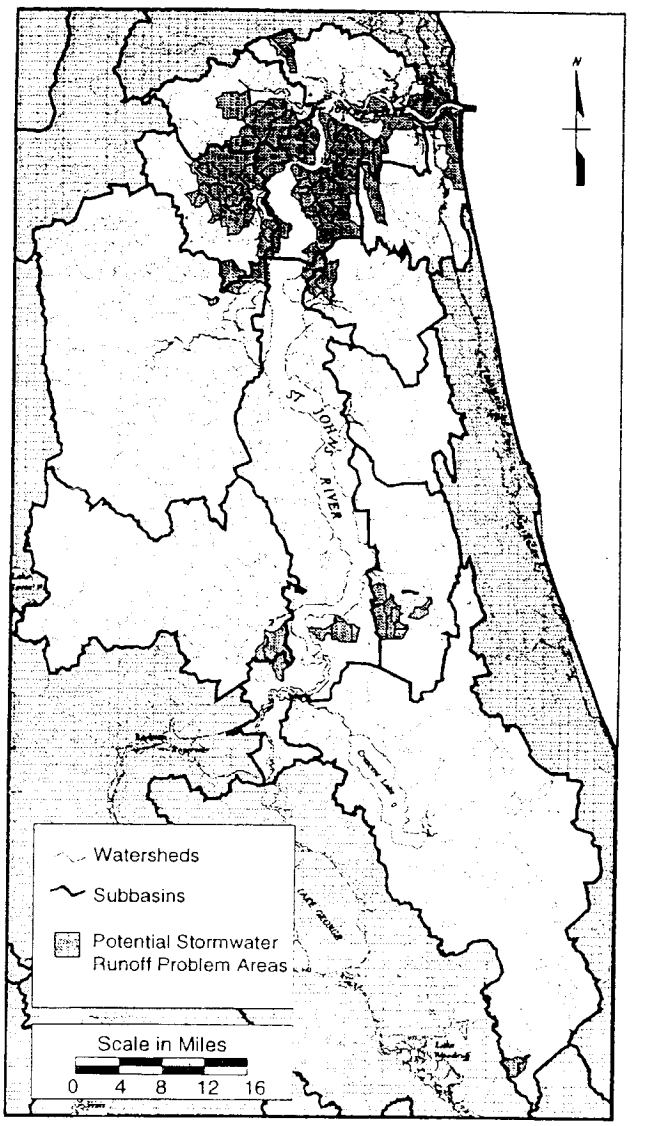

Figure 3. Potential stormwater runoff problem areas, lower St. Johns River basin.

county maps are guides to future development, and not predictions of actual development, we exercised caution when using the results. Problem areas were defined as those watersheds with projected loads greater than or equal to existing problem areas. Also, District planners combined model results with information about individual counties' regulations and policies to evaluate where problems are most likely to occur.

Prior to compiling the Plan, the District conducted workshops in each county in the District, in which problem areas identified by the PLSM were discussed with local agency staff, officials, and the public. We provided large, hard copy maps depicting stormwater runoff problem areas combined with results of a separate water quality analysis on county-based maps. These maps proved to be powerful tools for initiating discussions and gathering feedback.

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In the Plan, stormwater runoff problem areas were reported for each of the 10 major drainage basins in the District. The information was also repackaged in a county-based format to create a quick reference for local agencies. District planners recommended strategies for addressing problems; these strategies vary as appropriate for each county. Examples include the need to assess compliance with existing stormwater permits, encourage stormwater reuse during the stormwater and consumptive use permitting processes, coordinate with municipalities that are implementing stormwater management plans, encourage and assist significantly affected municipalities to create stormwater utilities, and improve monitoring in problem areas that do not have sufficient water quality data.

In conclusion, the Watershed Assessment GIS project has proved to be useful not only to the St. Johns River Water Management District, but also to local governments. Large projects such as this could not be completed in a reasonable time without the use of GIS. Also, for ARC/INFO users who have been restricted to vector processing, the cell-based processing available in GRID is a powerful modeling tool.

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Watershed Stressors and Environmental Monitoring and Assessment Program Estuarine Indicators for South Shore Rhode Island

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Abstract

The U.S. Environmental Protection Agency has initiated the Environmental Monitoring and Assessment Program (EMAP), a nationwide ecological research, monitoring, and assessment program whose goal is to report on the condition of the nation's ecological resources. During the summers of 1990 through 1993, data were collected from approximately 450 sampling locations in estuarine waters of the Virginian Biogeographic Province (mouth of the Chesapeake Bay to Cape Cod). During this period, sampling stations were located in the coastal ponds and coastal area of south shore Rhode Island.

One objective of EMAP is to explore associations between indicators of estuarine condition and stressors in the watersheds of the sampled systems. Extensive watershed information for south shore Rhode Island is available in geographic information system (GIS) format. Watershed stressors along south shore Rhode Island were compared with EMAP indicators of estuarine conditions using GIS analysis tools. The indicator values for coastal EMAP stations (those offshore from coastal ponds) were associated with all of the aggregated south shore watershed stressors. The coastal pond indicator values were associated with stressors in the individual coastal pond watersheds. For the total south shore watershed, the major land use categories are residential and forest/brush land, followed by agriculture. Closer to the coast, residential land use is more prevalent, while further from the coast, forests/brush lands dominate. All coastal EMAP stations, with one exception, exhibited unimpacted benthic conditions, indicating no widespread problems. For the individual watersheds, the major land use categories are residential and forest/brush land. The population density (persons per square mile) shows an increasing trend from west to east. Impacted benthic conditions were observed at EMAP sampling sites in two coastal ponds. These two impacted benthic sites appear to be organically enriched.

Introduction

Since its inception in 1970, the U.S. Environmental Protection Agency (EPA) has had the responsibility for regulating, on a national scale, the use of individual and complex mixtures of pollutants entering our air, land, and water. The Agency's focus during this period centered primarily on environmental problems attributable to the use of individual toxic chemicals. Regulatory policy, while continuing to control new and historical sources of individual chemicals (i.e., "end of the pipe") and remediate existing pollution problems, will have to address the cumulative impacts from multiple stresses over large spatial and temporal scales.

In this decade, the focus of environmental problems, or "scale of concern," has shifted from point-source and local scales to regional and global scales. Concurrently, the focus has shifted from chemical to nonchemical stressors. The threat posed by nonchemical stresses (e.g., land use, habitat alteration and fragmentation, species loss and introduction) presents a substantial risk to the integrity of both specific populations and ecosystems, and entire watersheds and landscapes.

The shift in the scale of concern for environmental problems presents a unique challenge for environmental decision-making. Traditionally, environmental information has been collected over local spatial and short temporal scales, focused on addressing specific problems, limited in the number of parameters measured, and collected with a variety of sampling designs that were neither systematic nor probabilistic. It is not surprising, then, that several scientific reviews concluded that the information needed to assess, protect, and manage marine and estuarine resources was either insufficient or unavailable and recommended a national

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network of regional monitoring programs (1, 2). Two key recommendations resulted from these reviews: (1) the need for a national monitoring program designed to determine the status and trends of ecological resources, and (2) the need for an assessment framework for synthesizing and interpreting the information being produced in a timely manner and in a form that the public can understand and decision-makers can use. EPA's response to these recommendations was to institute a long-term monitoring program, the Environmental Monitoring and Assessment Program (EMAP), and to adopt a risk-based strategy for decision-making.

EMAP is a nationwide ecological research, monitoring, and assessment program whose goal is to report on the condition of the nation's ecological resources. During the summers of 1990 through 1993, data were collected from approximately 450 sampling locations in estuarine waters of the Virginian Biogeographic Province (mouth of the Chesapeake Bay to Cape Cod) (3-5). During this period, some of the sampling stations were located in the coastal ponds and coastal area of south shore Rhode Island. One objective of EMAP is to explore associations between indicators of estuarine condition and stressors in the watersheds of the sampled systems. Extensive watershed information for south shore Rhode Island is available in geographic information systems (GIS) format.

The intent of this paper is to compare watershed stressors with EMAP indicators of estuarine condition along south shore Rhode Island using GIS analysis tools. The indicator values for coastal EMAP stations (those offshore from coastal ponds) are associated with all of the aggregated south shore watershed stressors. The coastal ponds indicator values are associated with stressors in the individual coastal pond watersheds. The project reported on in this paper served as a pilot for integrating watershed information with wide-scale ecological data collected to assess condition of estuarine waters.

Ecological Risk Assessment Context

Robert Huggett, EPA's Assistant Administrator for Research and Development, is using the risk assessment-risk management paradigm as a framework to reorganize the EPA research laboratories (6). Huggett is also reorienting the research that EPA conducts to be risk based (both human and ecological). The major thrust of the research to be conducted in the EPA laboratories will be directed toward reducing the uncertainties in the risk assessment process. In this way, the risk assessment context provides the "why" for the research conducted.

Ecological risk assessment is defined as a process for evaluating the likelihood that adverse ecological effects have occurred, are occurring, or will occur as a result of exposure to one or more stressors (7). The value of the risk assessment framework lies in its utility as a process for ordering and analyzing exposure and effects information, and in its flexibility for describing past, present, and future risks.

One way of depicting the ecological risk assessment process is shown in Figure 1 (8). The key points are that the process is continuous; the process can be oriented in either direction, dependent upon the form of the question or issue being addressed; and monitoring is at the hub, providing information to all activities. The end result of the effort is to provide better information for making environmental management decisions.

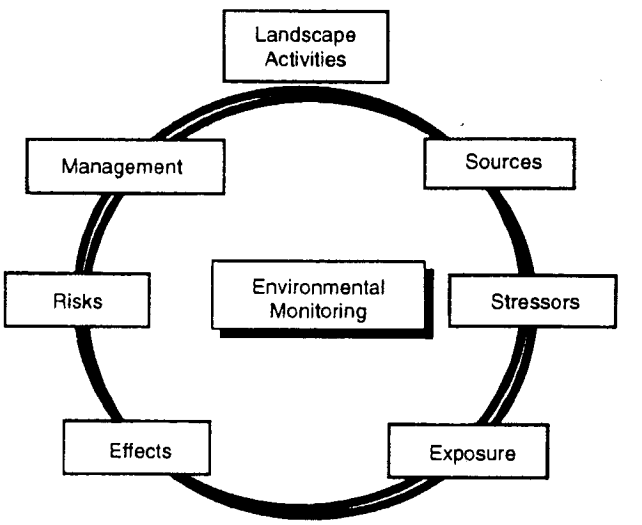

Figure 1. Ecological risk assessment framework (8).

Overview of EMAP and Estuarine Results

EMAP has been described as an approach to ecological research, monitoring, and assessment (9). It is not the only approach but is an approach that is driven by its goal to monitor and assess the condition of the nation's ecological resources. The objectives of the program to address this goal are to:

- Estimate the current status, trends, and changes in selected indicators of the condition of the nation's ecological resources on a regional basis with known confidence.
- Estimate the geographic coverage and extent of the nation's ecological resources with known confidence.
- Seek associations among selected indicators of natural and anthropogenic stress and indicators of ecological condition.
- Provide annual statistical summaries and periodic assessments of the nation's ecological resources.

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The approach used by the program to meet its objectives and address its goal includes:

- Use of a large, regional scope that encompasses the entire county but provides information on the scale that federal and regional environmental managers require.
- Emphasis on ecological indicators to provide the information to assess condition (i.e., collect information on the ecological systems themselves to determine their condition or "health").
- A probability-based sampling design to produce statistically unbiased estimates on condition and to provide uncertainty bounds for these estimates.
- A vision of the program as long-term, continuing into the next century, which is consistent with the large, regional spatial scale being addressed.
- Development through partnerships with other agencies that have natural resource stewardship responsibility.

The estuarine component of EMAP was initiated in 1990, with monitoring in the estuarine waters of the Virginian Biogeographic Province (mouth of Chesapeake Bay northward to Cape Cod) (10). Figure 2 depicts the biogeographic provinces of estuarine resources of the country. These provinces have been delineated based upon major climatic zones and the prevailing offshore currents (11). This is comparable with the ecoregion approach used to describe the distribution of terrestrial ecosystems (12). The biogeographic province is the comparable approach for coastal ecosystems. Monitoring in the Virginian Province continued through 1993; monitoring was conducted in the Louisianian Province from 1991 to 1994; monitoring was initiated in the Carolinian Province in 1994; and monitoring will be initiated in the West Indian Province in 1995.

A suite of measurements was collected at each of the EMAP-Estuaries sampling sites that were selected with a probability-based sampling design (13, 14). As indicated above, the measurements emphasized ecological conditions indicators, which included biotic indicators such as benthic and fish abundance, biomass, diversity, and composition, and also included abiotic indicators such as dissolved oxygen, sediment contaminant concentration, and sediment toxicity (15).

In the Virginian Province, approximately 450 probability-based sampling sites were visited during the summer periods in 1990 through 1993 using consistent indicators and collection and analysis procedures. An example of the results is shown in Figure 3, which presents the condition of benthic resources (16). The benthic condition is reported using a benthic index, which is an aggregate of individual benthic measurements that were combined using discriminant analysis to differentiate impacted from unimpacted sites (3, 17). The figure presents results for values of the benthic index that were determined to be impacted. The bar chart is the standard EMAP format for results: province-scale results with 95-percent confidence intervals about estimates. The large, small, and tidal categories refer to the strata used in the probability-based sampling design: large systems are the broad expanses of water such as in Chesapeake Bay, Delaware Bay, and Long Island Sound; small

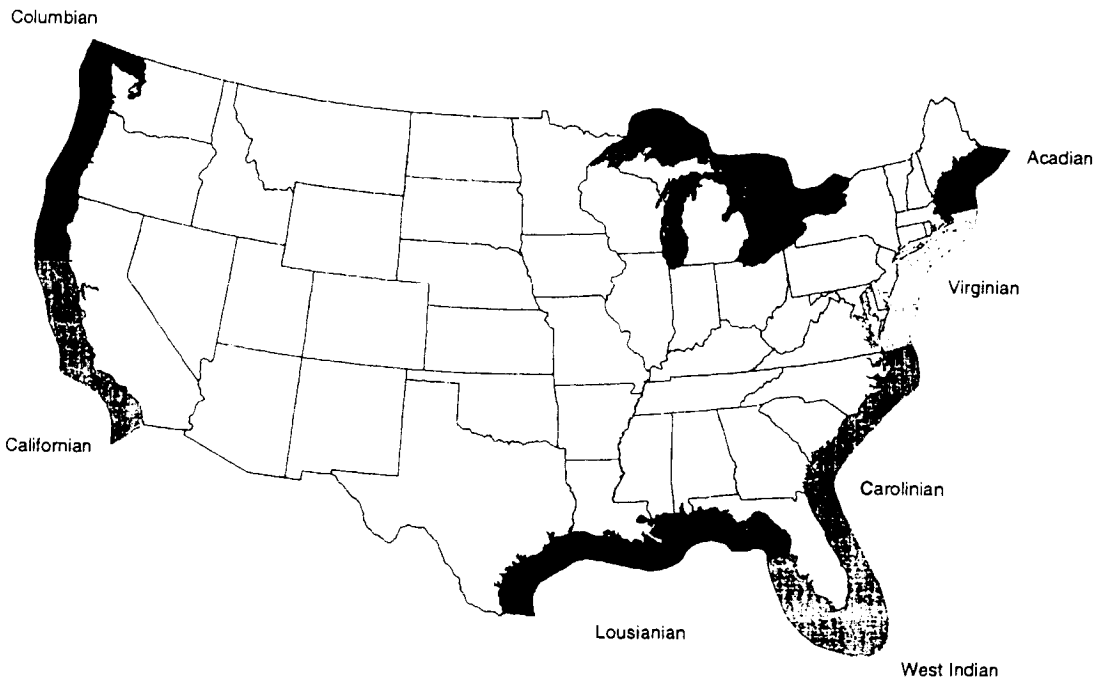

Figure 2. Biogeographic provinces used by EMAP-Estuaries (13).

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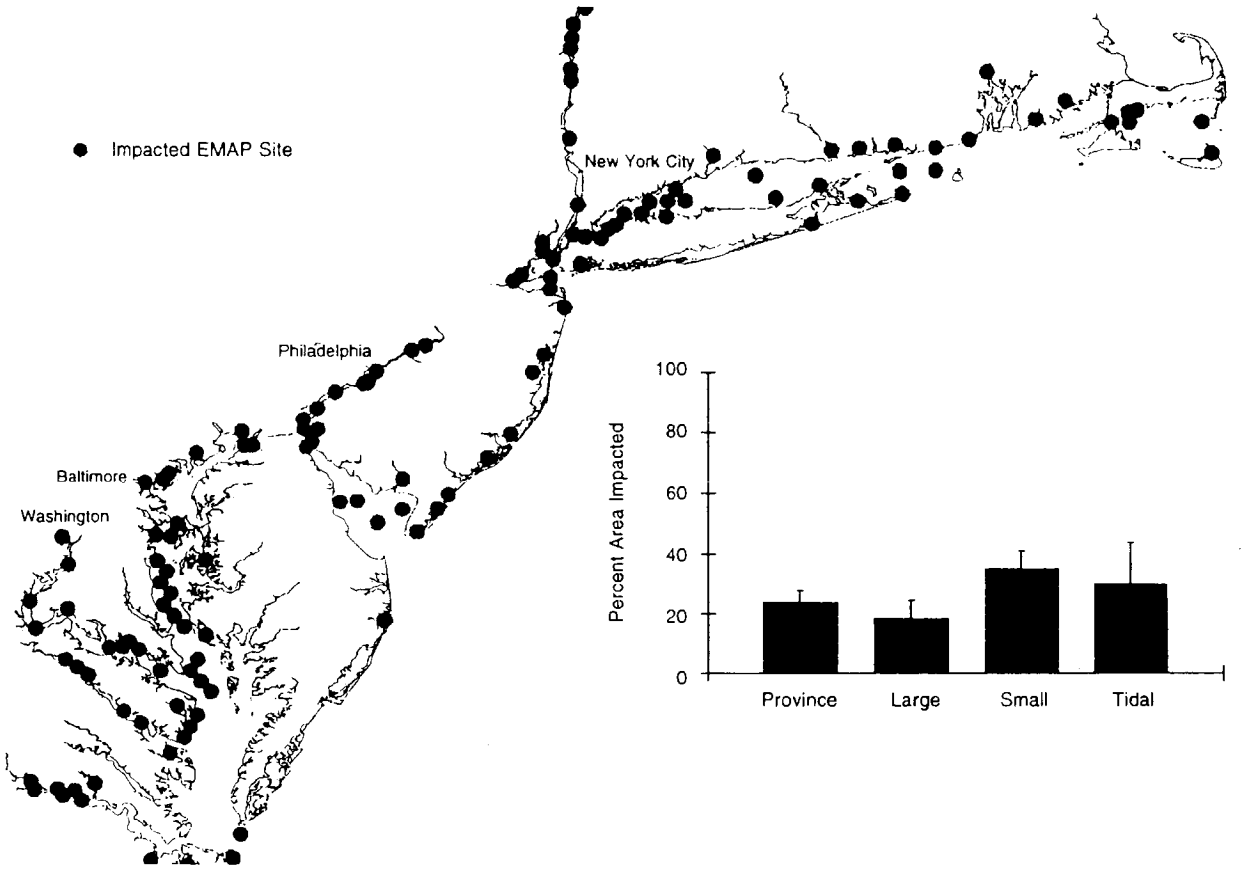

Figure 3. Condition of benthic communities in Virginian Province.

systems include the bays and harbors along the edges of the major systems and embayments along the coast; and large tidal rivers include the Potomac, James, Rappahannock, Delaware, and Hudson Rivers.

The results indicate that 24 percent ± 4 percent of the estuarine waters of the Virginian Province have impacted benthic communities. The small and tidal river systems have proportionately more impacted area than the large systems.

All the EMAP data are geographically referenced; therefore, the data can be spatially displayed to explore patterns. The spatial display of the impacted benthic community information is a simple spatial analysis of the EMAP data. This analysis shows that the impacted benthic resources are distributed across the entire province, with more impacted sites in the vicinity of the major metropolitan areas.

In addition to analyzing the EMAP results at the regional scale, analyses have been conducted at the watershed scale (see Figure 4). The probability-based sampling design permits the data to be aggregated (poststratified) in ways other than the way the original design was stratified. The only restriction to the aggregation is the number of available sample sites for the aggregation; a small number of sites leads to large uncertainties in the results. Figure 4 shows the aggregation for four major watersheds: Chesapeake Bay, Delaware Bay, Hudson-Raritan system, and Long Island Sound. This watershed scale is close to the practical scale at which environmental management decisions are implemented. The data need to be analyzed at smaller scales, however, to focus on environmental management of the smaller watersheds (e.g., contaminated sediments). This leads into the need to conduct the pilot project addressing watershed information.

South Shore Rhode Island Pilot Project

EMAP's third objective relates to exploring associations between indicators of estuarine condition and watershed stressors. Note that the word "watershed" was added. One way to address environmental management remediation strategies is to look at the watershed activities that could possibly be modified or changed to improve estuarine conditions.

Watershed stressors for the estuarine environment include land-based sources of pollution, such as point sources of pollution, and land use activities (i.e., how the land is actually used, including landscape patterns).

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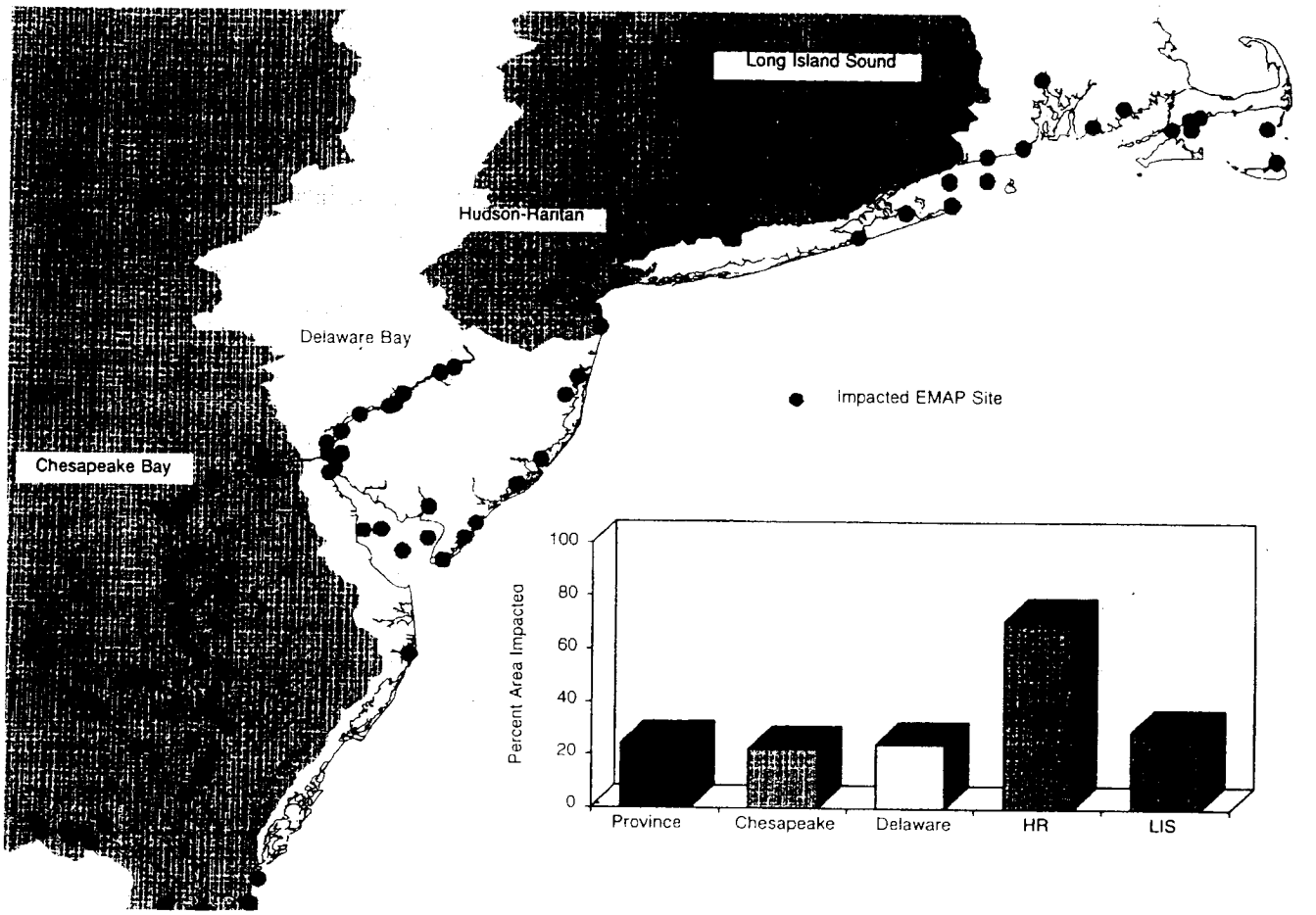

Figure 4. **Condition of benthic communities in major watersheds in Virginian Province (Chesapeake Bay, Delaware Bay, Hudson-Raritan system, and Long Island Sound).**

Which of these stressors is more important for a particular situation depends on the types of estuarine impact (localized or systemwide) and the management question that is being addressed.

The specific objective of the south shore Rhode Island pilot project was to compare watershed stressors with EMAP indicators of estuarine condition using GIS analysis tools. This project was not intended to be a definitive study by itself of south shore Rhode Island but to explore the process necessary to undertake the comparisons, to investigate the feasibility of pulling the necessary information together, and to identify potential problems before undertaking this comparison on a much larger scale.

The south shore Rhode Island study area is depicted in Figure 5. This coastal area drains into the coastal waters of Block Island Sound. The project was intentionally restricted to a limited geographic area to avoid being overwhelmed with the tremendous volumes of data that could have been encountered.

All data sources used in this project were available electronically. Digitized U.S. Geological Survey quad maps were available from the Rhode Island Geographic Information System (RIGIS) at the University of Rhode Island. The National Pollutant Discharge Elimination System (NPDES) was available for major dischargers from the National Oceanic and Atmospheric Administration's National Coastal Pollution Discharge Inventory (18). The 1990 census was also available from RIGIS. The EMAP 1990 through 1993 estuarine data were available from the EMAP-Estuaries Information System at the EPA Environmental Research Laboratory in Narragansett, Rhode Island. The RIGIS data were already available as ARC/INFO coverages. The NPDES and EMAP data had to be converted to ARC/INFO point coverages.

Two approaches were used to conduct spatial analyses. Buffer zones at 1, 3, 5, 10, and 20 kilometers from the south coast of Rhode Island were created and used to clip the south shore area coverages (e.g., land use, population, point sources). The watershed boundaries of three south shore coastal ponds were manually delineated and used to clip the south shore area coverages. The ponds were Quonochontaug, Ninigret, and Point Judith (west to east).

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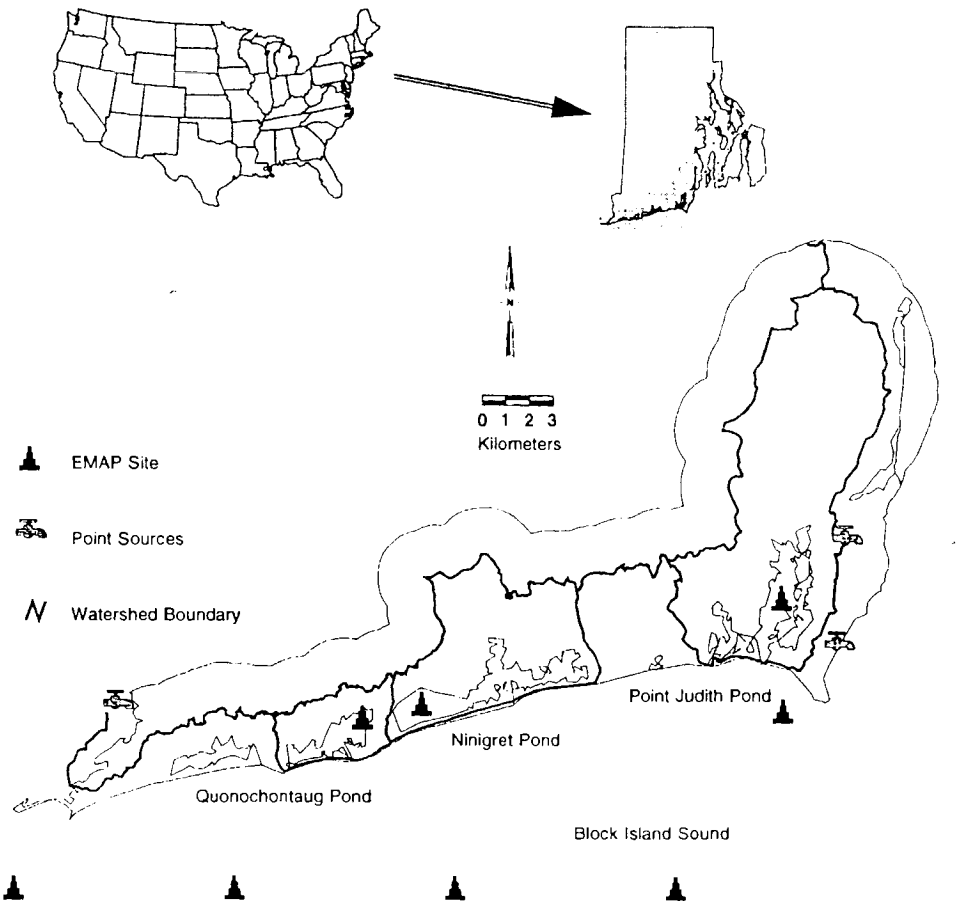

Figure 5. South shore Rhode Island study area.

South Shore Rhode Island Pilot Project Results

The results for land use by distance from the coast are presented in Figure 6. These results give the broad-scale coastal perspective. For the total south coast watershed, the major land use categories are residential and forest/brush land, followed by agriculture. Closer to the coast, residential land use is more prevalent, while farther from the coast, forests/brush lands dominate. Population (see Figure 7) increases with distance from the coast, but population density does not appear to be a function of distance from the coast. Only one out of five coastal EMAP stations exhibited impacted benthic conditions, indicating no widespread benthic problems in the coastal waters. The one station that was classified as impacted was dominated by an extremely high number of individuals of one species.

A smaller scale view can be gained by looking at the results for the individual watersheds. This view also provides an east-west perspective compared with the south-north perspective with the distance from the coast. Again, for the individual watersheds, the major land use categories are residential and forest/brush land (see Figure 8). The population increases from west to east, and population density shows an increasing trend from west to east (see Figure 9). Impacted benthic conditions were observed at the EMAP stations in Quonochontaug and Point Judith Ponds. These stations exhibited organic enrichment (total organic carbon in the sediments exceeding 2 percent), possibly from historically improperly treated sewage. No benthic data were available for the Ninigret Pond station; however, dissolved oxygen was observed to be low at this station. No major NPDES point sources are located in the coastal pond watersheds, although two are located on the eastern edge of the Point Judith Pond watershed boundary.

Discussion

A pilot project was conducted for south shore Rhode Island to compare watershed stressors with EMAP indicators of estuarine condition. The results indicate that such a comparison can be accomplished, with the watershed information providing a qualitative link to the estuarine conditions observed. One potential problem is the need to delineate the watershed boundaries for all

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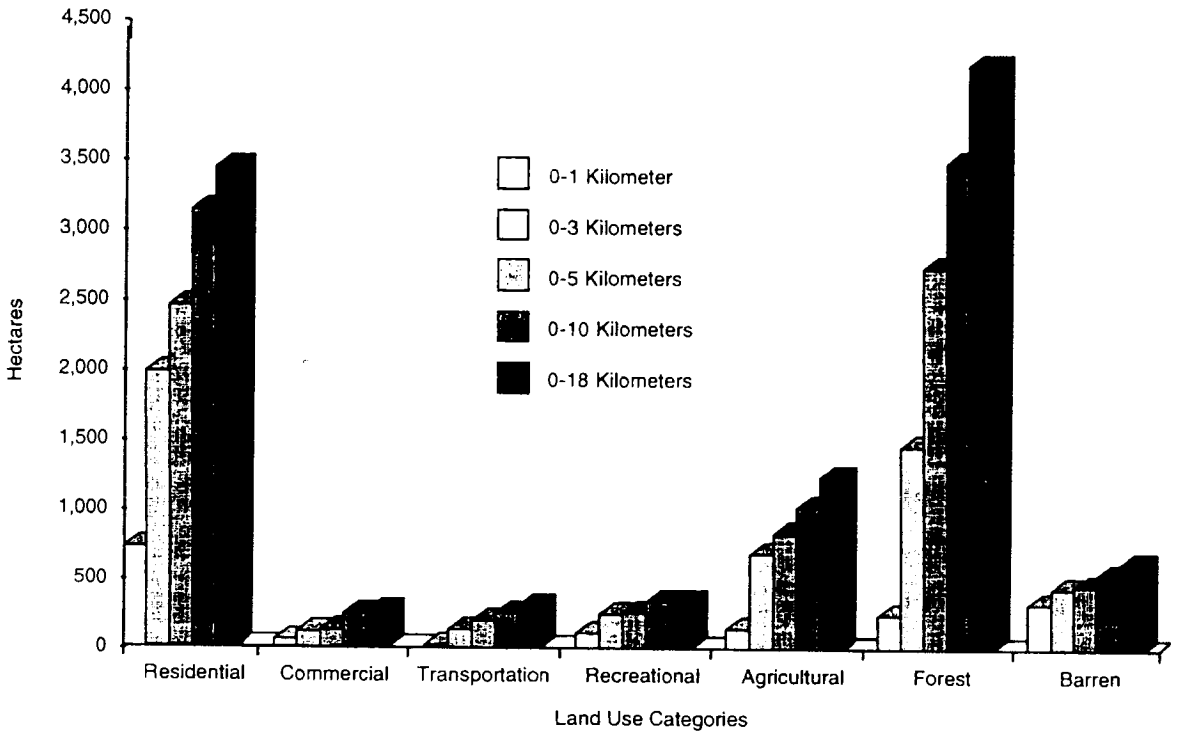

Figure 6. South shore Rhode Island land use by distance from south coast.

watersheds for which EMAP data are available. ARC/INFO provides tools for doing this, but practical application indicated that difficulties are encountered when the topographic relief is relatively flat, and on-screen corrections needed to be applied (19).

A restriction that needs to be understood before applying the procedures used in this project to a much wider geographic area is that the data sets for the watershed stressors need to be available over the wider geographic area. Further, these data sets need to be temporally consistent and constructed with consistent methods and land use classification schemes.

This project was conducted with only a small number of actual EMAP sampling sites, particularly for the individual watersheds. Because of this restriction, no statistical analyses were conducted with the EMAP data for comparison with the watershed information. Only qualitative comparisons were attempted. The next step is to increase the number of individual watersheds so that a rigorous statistical analysis can be conducted. This is being conducted by Comeleo et al. for comparing watershed stressors for subestuary watersheds in the Chesapeake Bay with estuarine sediment contamination (19). The steps after this will be to (1) apply the techniques to the entire EMAP Virginian Province data

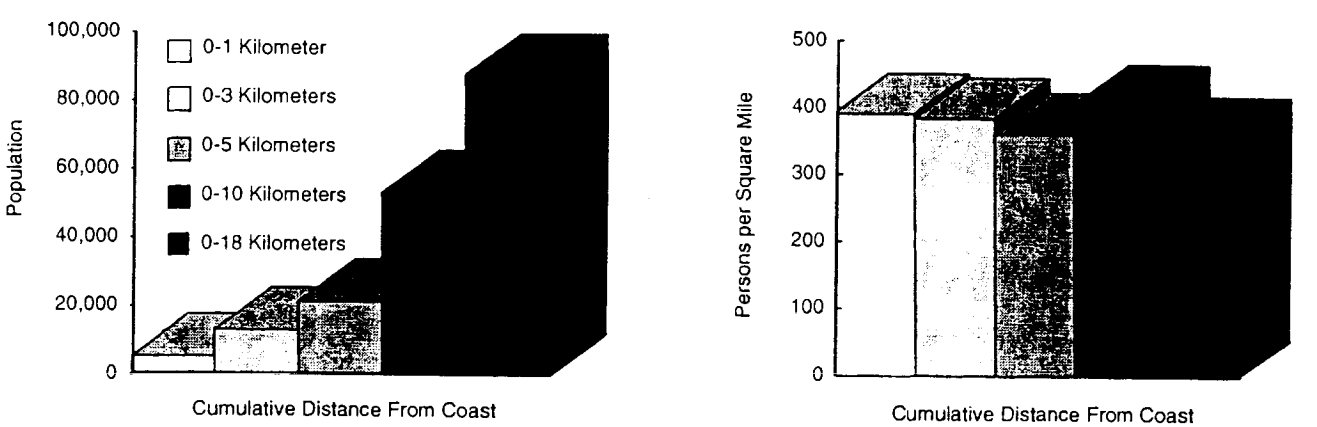

Figure 7. South shore Rhode Island population by distance from south coast.

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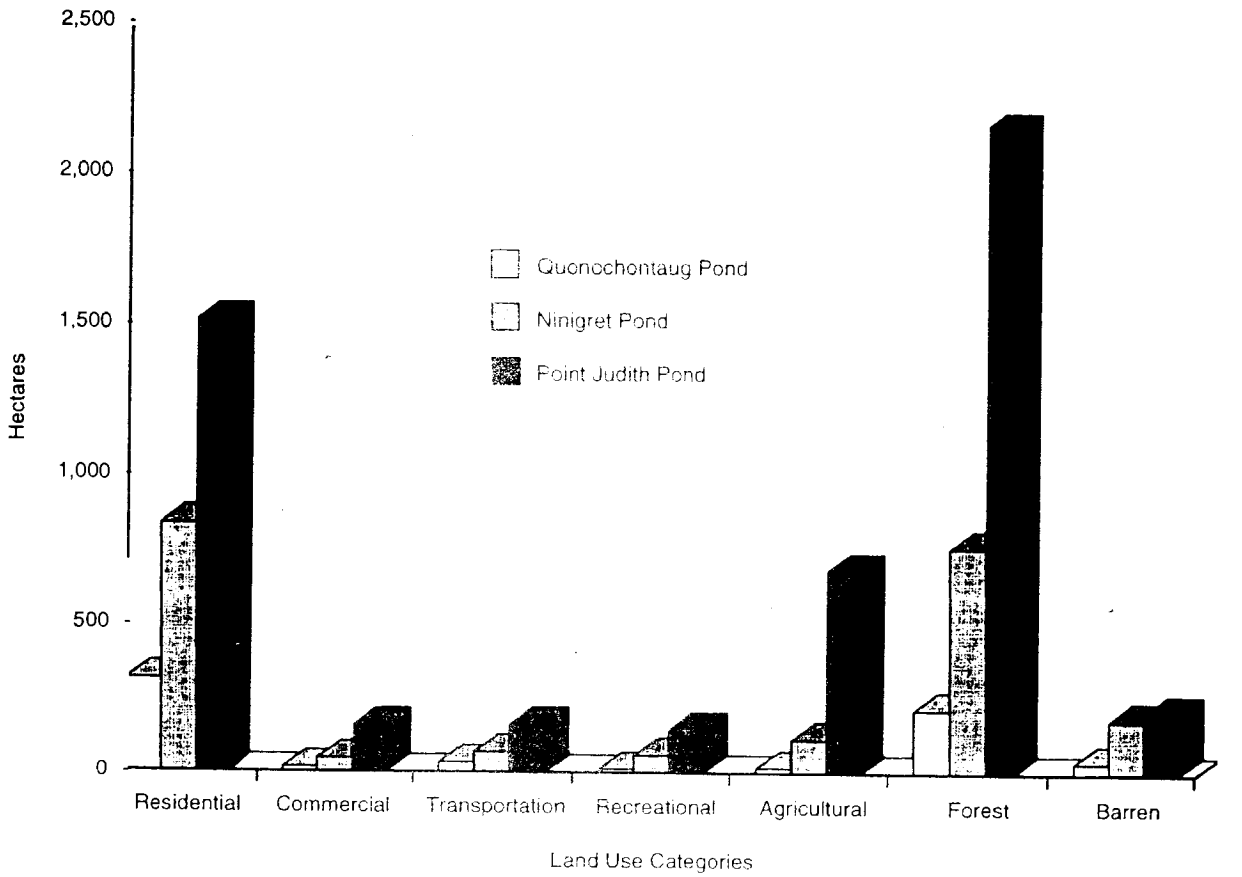

Figure 8. South shore Rhode Island land use by individual watershed.

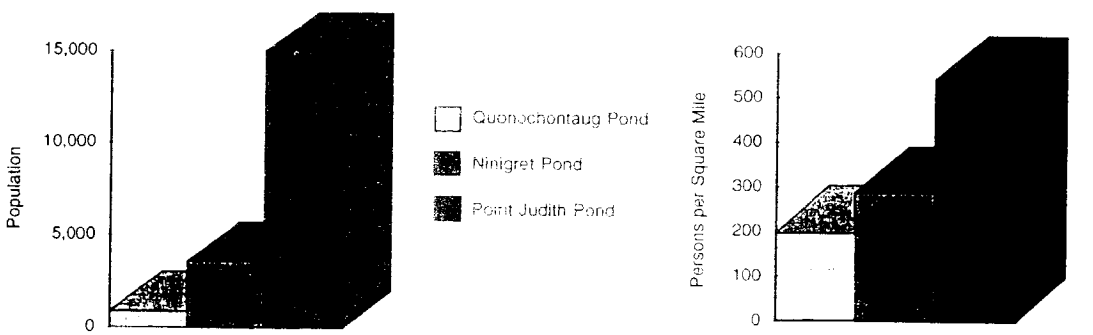

Figure 9. South shore Rhode Island population by individual watershed.

set, and (2) relate the watershed stressor information to estuarine benthic condition.

Acknowledgments

We thank S. Hale, J. Pesch, J. Copeland, and D. Keith for their review of this paper. This is contribution No. 1648 of the Environmental Research Laboratory-Narragansett.

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GIS Watershed Applications in the Analysis of Nonpoint Source Pollution

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Introduction

Geographic information systems (GIS) have been used to evaluate the impact of nonpoint source (NPS) pollution in a variety of watersheds and drainage systems over the past 20 years (1-6). During that period, our understanding of the sources and hydrologic transport mechanisms of NPS pollutants, both in particulate and soluble forms, has greatly increased (7-9). Our ability to create and manipulate land resource data, however, has advanced at a far more dramatic rate. Whereas 20 years ago, both computer system capabilities and peripheral hardware limited the process of encoding, storing, and displaying spatial data, today we can encode land resource data, analyze it, and produce stunning visual displays at a relatively low cost.

The question is: what has this experience told us regarding the yet unresolved problem of water quality degradation from NPS pollution in our streams, lakes, and coastal waters (10, 11)?

The purpose of this paper is to report on several recent studies of this nature that created a GIS as a tool to analyze NPS pollution. This paper will not cover all aspects of these studies; detailed reports on each project are available from the authors or respective clients. The objects of these studies were:

- A medium-sized lake draining a fairly small watershed
- A riverine system with multiple use impoundments
- A 100-mile stretch of Atlantic coastal estuary

These water bodies all have one common ingredient: NPS pollution significantly affects them. While the primary focus of these studies was to understand the dynamics of surface water quality, and specifically the NPS component, the further objective was to document the causal link between identified water resource problems and the watershed-wide management actions needed for their remediation. Thus, GIS serves not only as a mechanism for analysis of NPS pollution sources but also as the tool by which to evaluate alternative methods that would reduce or prevent this pollution.

Study Concepts

These three studies illustrate different approaches to both aspects of this problem. In the 93-square-mile Upper Perkiomen Creek watershed (UPW) study, the objective was to develop a management program that would reduce nutrient load in a system of reservoirs at the base of the watershed. An essential element in the analysis underlying GIS design (ARC/CAD) was to be able to differentiate and evaluate pollution sources in the watershed, while providing the technical basis for an innovative and far-reaching management program on all levels of government; that is, GIS was used not only to analyze the problem but to help formulate the solution.

In the more focused Neshaminy Creek study, Cahill Associates (CA) designed a detailed pixel/raster format for GIS to support detailed hydrologic modeling (12) and NPS loading analysis. This study, carried out under Pennsylvania's Act 167 stormwater management program, was under a legal requirement to translate technical findings into subdivision regulations that all 30 watershed municipalities would adopt. This mandate required much more geographically specific rigor in the GIS approach and in the management recommendations the law stipulated.

These two projects (see Figure 1), when taken together, illustrate the critical relationship between understanding the appropriate level of detail in GIS system design, GIS development with modeling and other analytical requirements, and ultimately, the proposed management actions for watershed-wide implementation.

In the New Jersey Atlantic Coastal Drainage (ACD) study, the objective was to document more completely the magnitude and sources of NPS pollutants, especially nutrients, entering New Jersey estuarine coastal waters. The GIS design placed special attention on the role of urban or developed land uses situated along the coastal fringe, particularly the maintained or landscaped portions of developed sites. Most previous studies have largely ignored this factor. Instead, they have focused water quality analysis typically on NPS loadings as a

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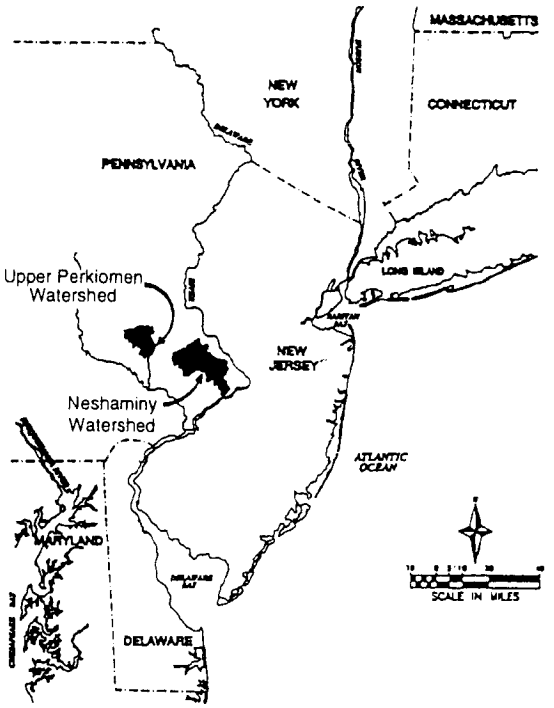

Figure 1. Regional location of Upper Perkiomen and Neshaminy basins.

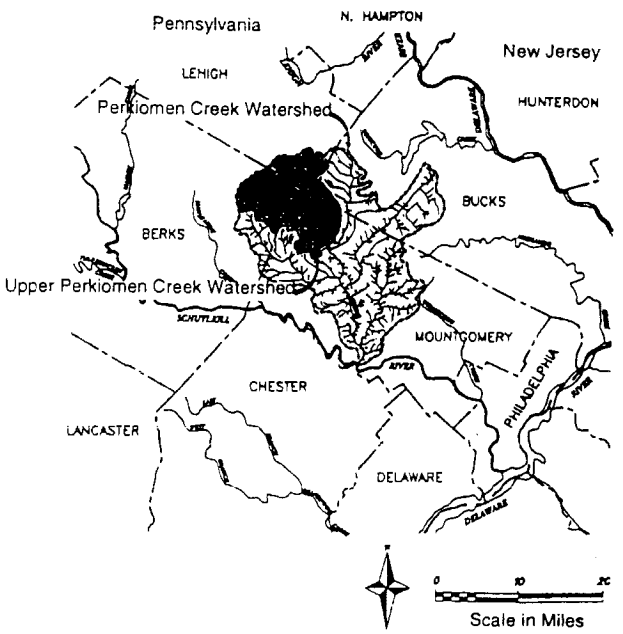

Figure 2. The Perkiomen Creek watershed in the Delaware River basin.

function of impervious area coverage, with the assumption that loadings increase as imperviousness increases.

On the contrary, the CA thesis states that certain pollutant loadings, such as nutrients, maximize in areas with relatively moderate densities (1/2- to 1-acre lots) and percentage impervious cover but with large maintained lawnscapes. Because sandy soils allow soluble NPS pollutants to pass as interflow to points of surface discharge with surprising ease, they exacerbate the problem of nutrient applications in typical coastal drainage areas. GIS application in this case enabled estimation of the nutrient loading to coastal waters. Existing fertilized lawn areas were calculated to be a significant source of nutrient pollution, with loadings from new land development posited as an even more serious problem for New Jersey's coastal waters. GIS was then applied to evaluate the suitability of various best management practices (BMPs), based on the physical and chemical properties of the soil mantle and the existing and anticipated land use.

The Upper Perkiomen Creek Watershed Study

Background

The UPW in southeastern Pennsylvania is a tributary of the Schuylkill River in the Delaware River basin (see Figure 2). Serious eutrophication problems occurring in the system of reservoirs lying at the base of this relatively rural watershed prompted the study. The study effort evolved from concerns on the part of the Delaware Riverkeeper, a private nonprofit environmental organization dedicated to promoting the environmental well-being of the Delaware River watershed. The Upper Perkiomen Creek has experienced various water quality problems, especially the eutrophication of Green Lane Reservoir, a large raw water supply storage reservoir (see Figure 3). Green Lane's highly eutrophic condition has been a constant since shortly after initial construction over 35 years ago, but the relative importance of NPS inputs has dramatically increased. Whereas 10 years ago point source input was the major source of phosphorus, elimination of some point sources and advanced waste treatment for others has greatly reduced that component of pollutant loading, while NPS sources have remained constant or increased. Current analysis indicates that NPS pollution constitutes over 80 percent of the annual load of phosphorus (see Figure 4) into the Green Lane Reservoir and is well in excess of the desired loading to restore water quality (see Figure 5).

Nonpoint Source Analysis

Calculating the NPS load was an essential ingredient in the study and relied on developing accurate measurement of NPS transport during stormwater runoff periods. Certain pollutants, specifically those associated with sediment and particulate transport such as phosphorus, have produced a "chemograph" that parallels but does not exactly follow the traditional form of the hydrograph (see Figure 6). The pollutant mass transport associated with this runoff flux frequently constitutes the major fraction of NPS discharge in a given watershed (8, 13). In

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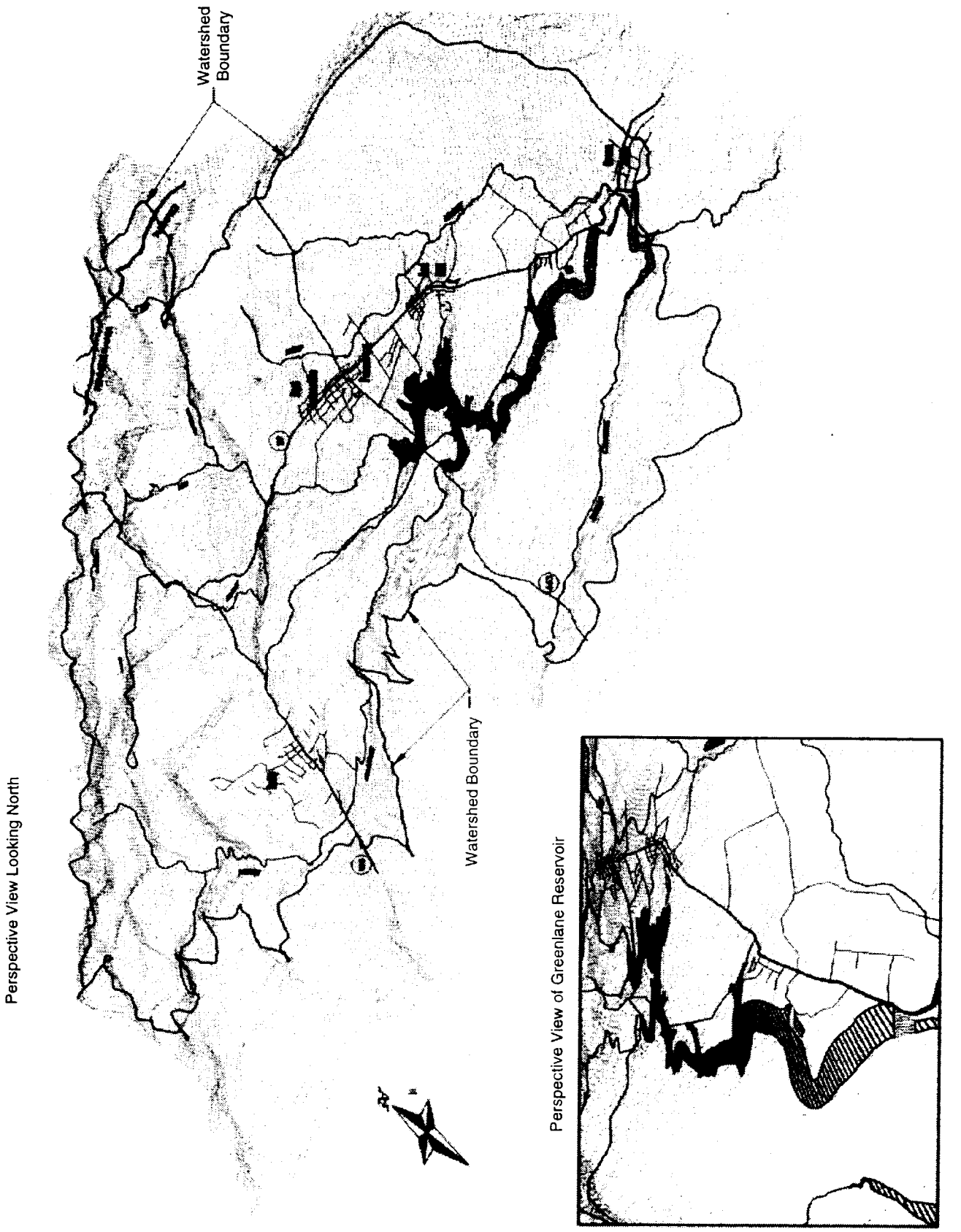

Figure 3. Green Lane Reservoir in the Upper Perkiomen watershed, 814 acres, 4.3 BG.

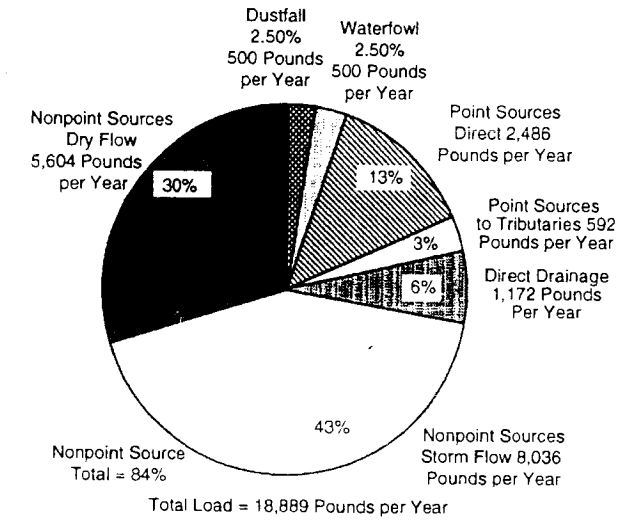

Figure 4. Sources of total phosphorus mass transport into the Green Lane Reservoir from the Upper Perkiomen watershed (71 square miles) in an average flow year—in pounds per year.

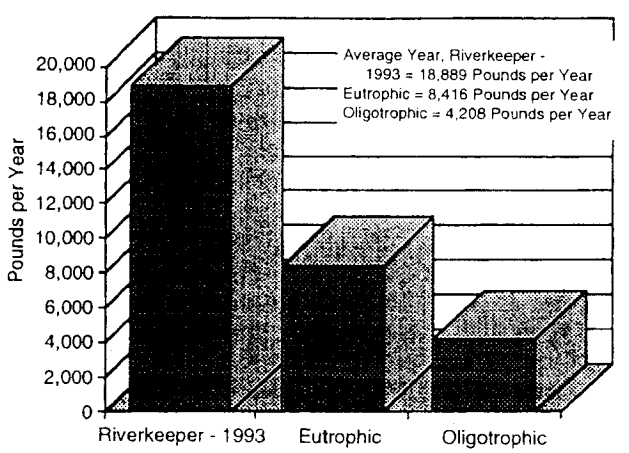

Figure 5. Reduction in annual phosphorus load required to achieve improved trophic level.

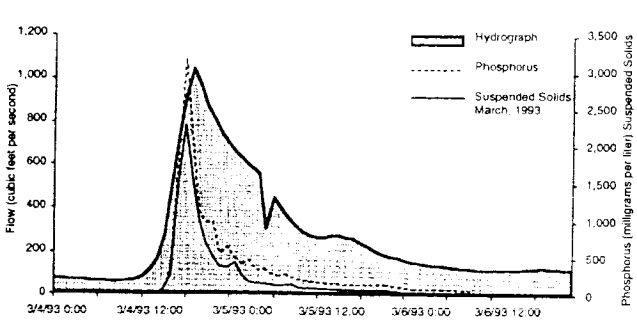

Figure 6. Storm hydrograph in the Upper Perkiomen watershed illustrating the dramatic increase in total phosphorus and suspended sediment during runoff.

the UPW study, operating continuous sampling stations at two key gage locations above the reservoir allowed the measurement of stormwater chemistry of this type and produced estimates of wet weather transport of phosphorus and sediment. Surprisingly, the NPS transport during dry weather, calculated by subtracting the point sources, was also significant and is attributed to livestock discharges and septage drainage.

But the wet weather proportion of NPS pollution still dominates lake water quality. Many have said that water quality in a given watershed is a function of land use, but that statement is as unsatisfying as saying that runoff is a function of rainfall. Experience has taught us that neither process is quite that simplistic, nor does either follow a direct linear relationship of cause and effect. The causal mechanisms that generate a certain mass load of pollutant in a drainage basin certainly result from how much mass of that pollutant is applied to the landscape within the drainage, which in turn is scoured from the landscape during periods of surface saturation, transported in, and diluted by runoff. The end result is a concentration of pollutant in the stormwater that might be several orders of magnitude greater than during dry weather flow, the hydrologic period traditionally used to measure and define water quality.

Developing NPS analysis or algorithms for stormwater quality modeling requires replicating the specific hydrograph and its associated chemograph, as well as defining the mechanisms by which pollutants are scoured from the land surface, transported in runoff, and pass through the river system. Total phosphorus (TP), for example, is transported with the colloidal soil particles (see Figure 7), so sediment transport and deposition constitute a key mechanism.

Adding to these complications is the question of whether to model single or multiple events. Is the chemodynamic process one in which the transport takes place over a series of storm events, so that each storm moves the pollutant mass a given distance in the drainage and then allows it to settle in the channel only to resuspend it with the next peak of flow? Or does the total mass transport occur in one single dynamic, from corn field or suburban lawn to lake, estuary, or other sink, that is hours or days downstream in the drainage? The issue of how stormwater transport of pollutants takes place is of paramount importance in current planning and regulatory implementation (11) because many of our current BMPs are relatively ineffective in removing NPS pollutants. This understanding is critical even as we attempt to intervene in the pollutant generation process by changing the way we cultivate the land, fertilize our landscapes, or for that matter, how we alter the land surface during growth.

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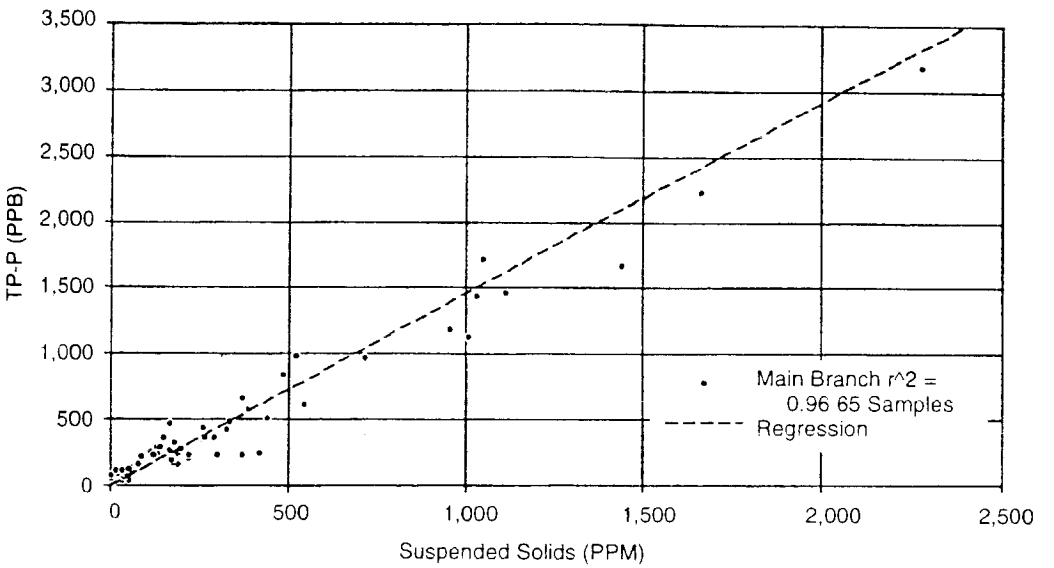

Figure 7. The relationship between total phosphorus and suspended sediment concentrations during runoff is strong but varies with different watersheds.

GIS Evaluation

The GIS data files on land use/land cover that were created for the UPW show that the bulk of the area is still quite undeveloped and rural (see Figure 8), with the steeply sloped and igneous rock areas in the headwaters in forest cover (38 percent) and the valleys in mixed agriculture (44 percent). The urban/suburban land composes the remaining 18 percent and largely consists of several older, historic boroughs linked together in a lineal pattern with widely scattered, low-density residential areas. Much of the existing housing is turn-of-the-century at quite high densities, mixed with a variety of commercial and other uses. This pattern contrasts sharply with typical large-lot suburban subdivisions. In fact, these watershed boroughs resemble the "village" concepts that innovative planning theorists advocate in a variety of important ways.

The watershed (see Figure 9) is blessed, or cursed, depending upon one's perspective, with a multiplicity of local governments including four different counties and 18 different municipalities. This arrangement poses special challenges for management program implementation. Population projections indicate that additional development will occur at moderate rates throughout the watershed, reflecting recent trends.

Farming, both crop cultivation and dairying, is a major existing land use in the watershed, although agriculture is not especially robust and appears to be declining. This lack of agricultural vibrancy becomes a major factor in determining how to impose additional management measures on agricultural pollution sources. GIS tabulation of agricultural land totals some 19,000 acres above the reservoir, which can be compared with the estimated TP and suspended solids (SS) mass transport reaching the lake. Considering only the agricultural land to be the source of this NPS input (not quite true) suggests an average annual yield of 180#/acre/year-SS and 0.22#/acre/year-TP.

This sediment/phosphorus yield is more than sufficient to maintain a eutrophic condition in the reservoir system. The problem with this yield, however, is that it is two orders of magnitude less than commonly accepted methodologies of soil erosion, such as the universal soil loss equation (14), would suggest might come from such a watershed. Analysis of the cultivation practices taking place on farmland in the watershed estimates soil erosion to be approximately 5 to 10 tons per acre or more per year, far more than is observed passing out of the basin into the reservoir. The phosphorus applications on both cultivated and maintained residential landscapes also appear much greater than the mass transport actually measured in the flowing streams, which represent perhaps 7 percent or less of the annual land application.

The implication for NPS analysis is that the standard shopping list of either agricultural or urban BMPs might only reduce the mass transport by a relatively small fraction, even if successfully applied throughout the drainage. As Figure 7 illustrates, most of the phosphorus transport occurs on the colloidal fraction of sediment particles, which tend to remain in suspension as stormwaters pass through conventional detention structures, terraces, or grassed swales.

To consider more radical measures, GIS was used to determine possible alternatives, such as creating a stream buffer system (see Figure 10) with various setback distances from the perennial stream network, and to evaluate how great an impact this might have on agricultural land use and urban development. Land

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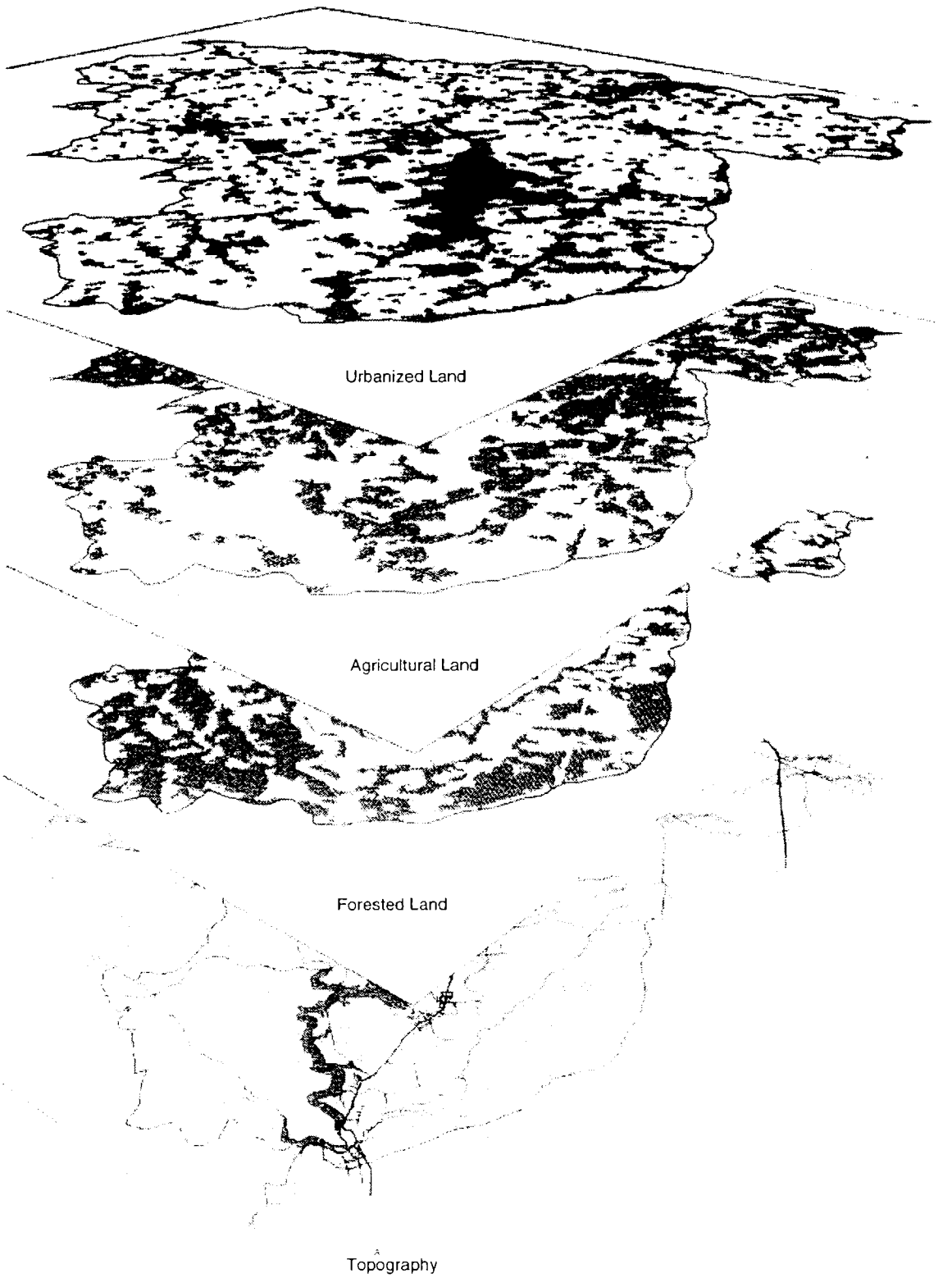

Figure 8. GIS data files showing land use/land cover characteristics for the Upper Perkomien watershed.

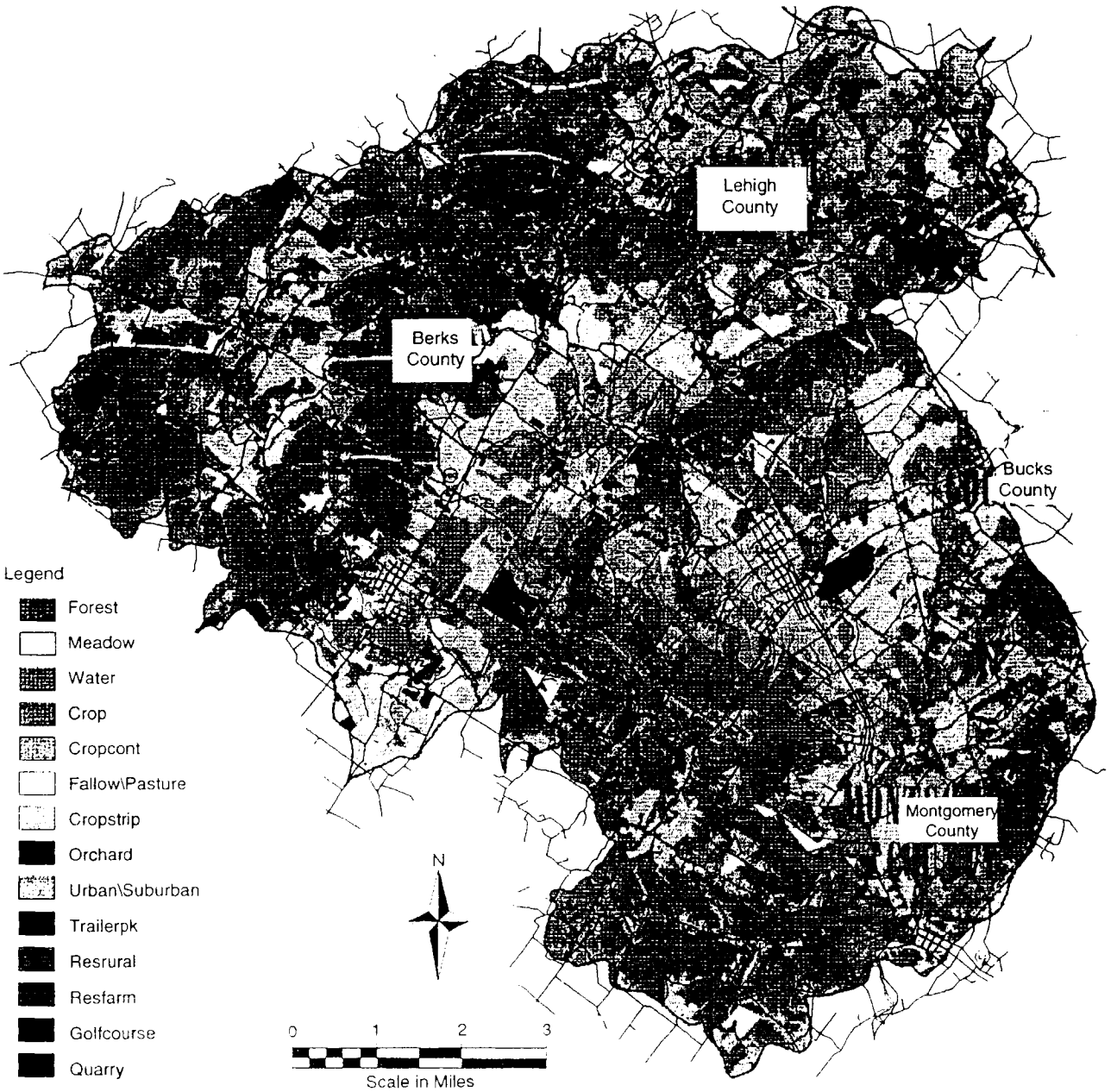

Figure 9. Existing land use/land cover GIS file for the Upper Perkiomen watershed. The 95-square-mile basin includes portions of four counties and 18 municipalities.

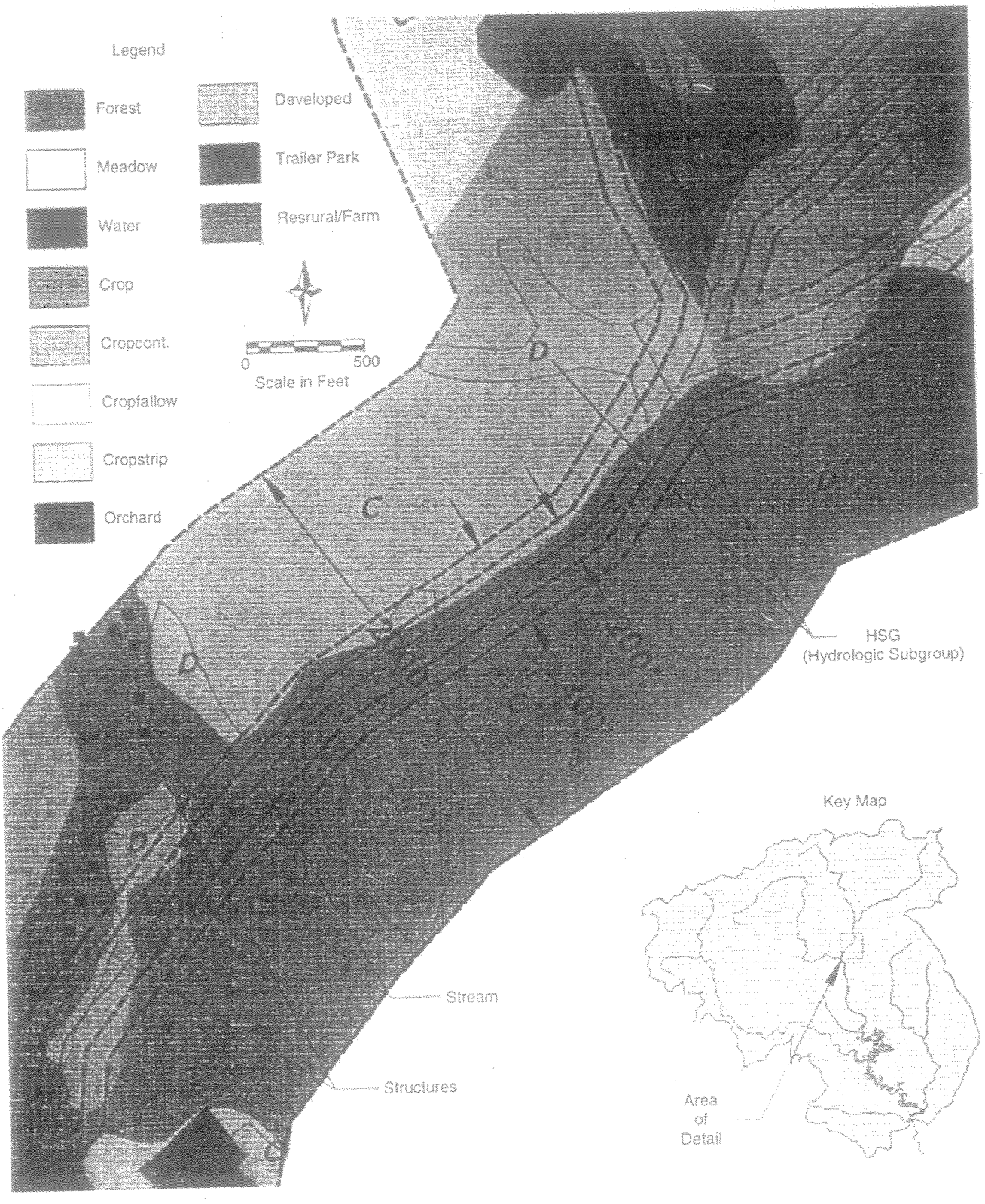

Figure 10. GIS analysis of stream corridors allows evaluation of riparian buffer systems, potential agricultural land loss, potential septic system discharges, and related NPS reduction with selected best management practices.

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use at varying distances (100 feet, 200 feet, and 1,000 feet) from streams was tabulated, including all land area in the "active" agriculture categories. This GIS documentation allowed estimation of the significant NPS reduction in loadings that a riparian corridor management program could achieve.

In the same way, GIS analysis helped estimate pollutant loadings from malfunctioning onsite septic systems. Counts of structures in nonpublicly sewered areas within varying distances from the stream system were developed using GIS data files. The nearly 300 potential systems within a 200-foot radius of those streams draining into the Green Lane Reservoir identified in this manner, with pollutant generation factors applied, became the basis of a dry weather pollutant estimation. Although this approach was dependent on a variety of assumptions, alternative approaches of evaluating the problem, such as field visits to actual onsite systems throughout the watershed, would not have been feasible.

For urban and suburban development, the management focus was to estimate NPS loadings from future land development. GIS was used to demonstrate NPS pollutant load implications of future growth envisioned in the watershed's keystone municipality, Upper Hanover Township. Here, an increase of 15,000 residents would convert 1,772 acres into residential, commercial, and industrial uses. Nonpoint pollutant loadings generated by this new land development constituted significant increases in phosphorus, suspended solids, metals, oil/grease, and other pollutants, and would reverse any improvements in Green Lane Reservoir water quality that recent wastewater treatment plant upgrades achieved.

From a water quality perspective, future alternative land use configurations that concentrate development and minimize ultimate disturbance of the land surface yielded would substantially reduce NPS pollutant loadings into the reservoirs. This entire process of testing land use implications of different management approaches for their water quality impacts indicated that pollutant loads could be minimized far more cost effectively through management actions, both structural and nonstructural, which varied from the areawide to the site-specific.

Neshaminy Creek Watershed Stormwater Management Study

Background

The Neshaminy Creek watershed, including 237 square miles of mixed urban and rural land uses, lies primarily in Bucks County, Pennsylvania, and flows directly into the Delaware River (see Figure 1). The 1978 Pennsylvania 167 Stormwater Management Act, which required that counties prepare stormwater management plans for all 353 designated watersheds in the state, mandated the Neshaminy study. This act further stipulated that municipalities then needed to implement the watershed plans through adopting the necessary municipal ordinances and regulations. In fact, the Neshaminy study had three water resource management objectives:

- Prevent worsened flooding downstream caused by increased volumes of runoff from land development.
- Increase ground-water recharge.
- Reduce NPS pollutant loadings from new development.

In the initial study design, water quality and NPS issues were secondary to flooding concerns. When Pennsylvania's stormwater management program was conceived, the state focused on preventing watershed-wide flooding. Clearly, detention basins have become the primary mode of managing peak rates of stormwater discharge site-by-site in most communities. Because detention basins only control peak rates of runoff and allow significantly increased total volumes of water discharged from sites, however, the increased stormwater volumes can theoretically combine and create worsened flooding downstream. Consequently, most Act 167 planning has focused on elaborate hydrologic modeling designed to assess the seriousness of potential cumulative flooding in watersheds under study.

In the case of the Neshaminy, however, the record suggested that although localized flooding could be an issue, an existing network of eight multipurpose flood control structures constructed during the 1960s served to prevent significant flooding. Water quality certainly was a serious stormwater concern, however, especially in the areas flowing into the reservoirs where recreational use had become intense. Several of the existing impoundments were multipurpose, their permanent pools providing critical recreational functions for a burgeoning Bucks County population. At the same time, the proliferation of development in the watershed, with its increased point and nonpoint sources, had degraded streams and seriously affected the reservoirs. While the total stream system in the watershed was of concern, the future of the reservoirs came to be particularly important in developing the total stormwater management program for the Neshaminy watershed.

The Neshaminy lies at the heart of Bucks County, Pennsylvania's primary population and employment growth county (see Figure 11). Although the Neshaminy watershed has already experienced heavy development, especially in the lower or southern portions, farmsteads and large areas of undeveloped land still exist, especially in headwater areas. Agriculture has been a major land use in the past, but farms rapidly are converting to urban uses as the wave of urbanization moves outward from Philadelphia and from the Princeton/Trenton metropolitan areas. Growth projections indicate continu-

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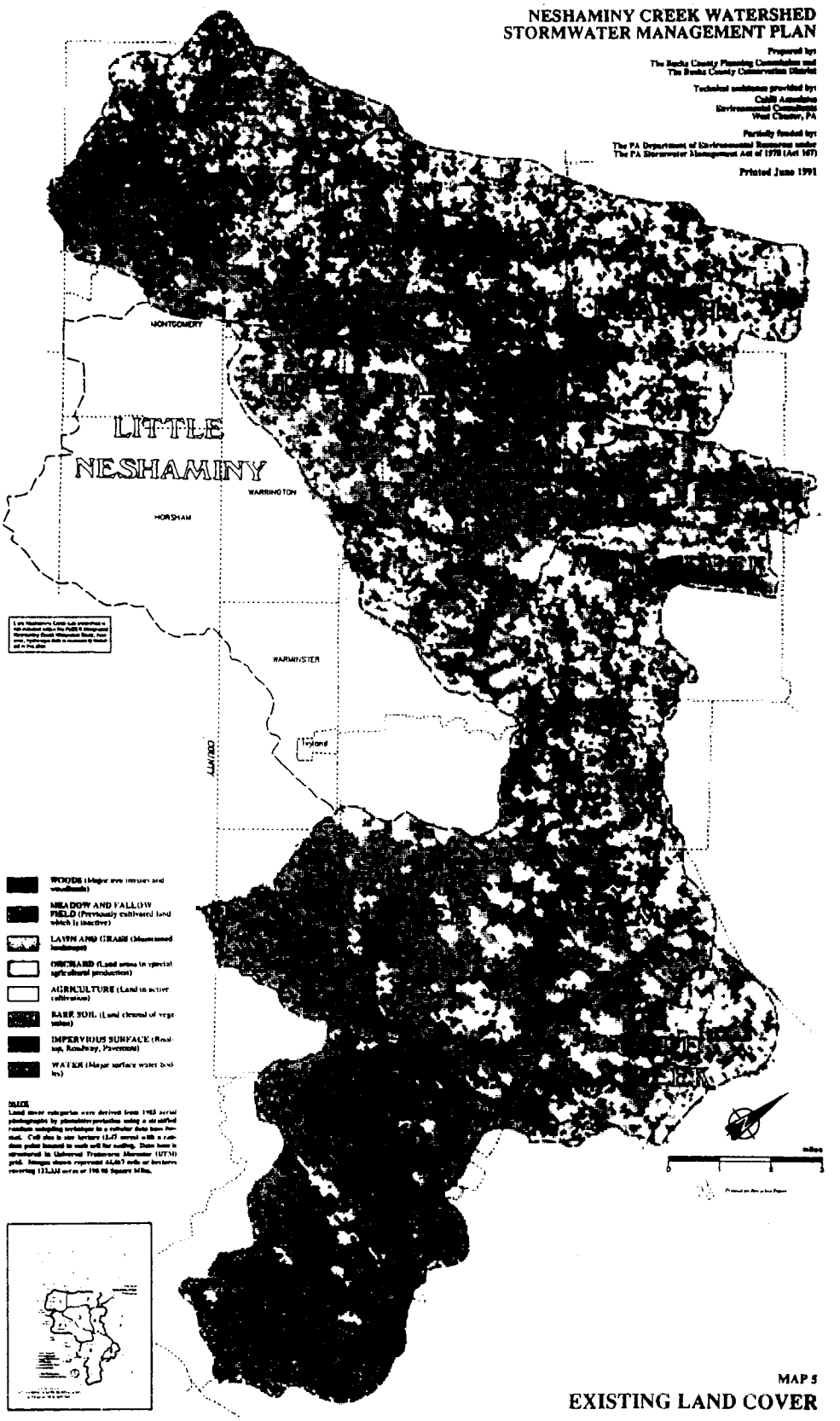

Figure 11. Land use/land cover in the Neshaminy basin of Bucks County, Pennsylvania. The watershed covers 237 square miles in southeast Pennsylvania.

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ation of this rapid growth and a continuing change in existing land use/land cover, together with projected development within the required 10-year planning horizon.

Physiographically, the watershed spans both the Piedmont and Atlantic coastal plain provinces, with rolling topography and relatively steep slopes underlain by Triassic formation rock, including the Lockatong, Brunswick, and Stockton formations. This bedrock ranges from being a poor aquifer (Lockatong) to an excellent aquifer (Stockton) where the many rock fractures allow for considerable ground-water yields. Soils are quite variable, ranging from good loam (hydrologic soil group B) to clays and other types with poor drainage characteristics (e.g., high water table, shallow depth to bedrock). A large proportion of the soils in the watershed are categorized as hydrologic soil group C, which is marginal for many stormwater management infiltration techniques (see Figure 12) and produces a relatively large proportion of direct runoff. With an annual rainfall of 45 inches, base flow accounts for about 12 inches and direct runoff accounts for 10 inches.

The system of eight stormwater control structures, which were built over the past three decades under the federal PL 566 program, have altered the hydrology of the watershed (15). In addition, in heavily developed portions of the watershed, impervious surfaces combined with numerous detention basins prevent the bulk of the precipitation from being recharged, and the volume of total runoff proportionally increases. An elaborate system of municipal and nonmunicipal wastewater treatment plants also adds to this alteration of the hydrologic cycle. These plants discharge wastewater effluent that, in some cases, constitutes the bulk of the stream flow during dry periods. While the impact of NPS was evident throughout the drainage, it was of special interest in the impoundment network, especially those impoundments that were conceived as multipurpose in function and constitute major recreational resources in the watershed.

GIS Design

Act 167 requirements and the needs of the hydrologic and other modeling used in planning both heavily influenced the GIS developed for the Neshaminy. Spatial data files, including existing land use, future land use, and soil series aggregated by hydrologic soil groups, were created by digitizing at a 1-hectare (2.5-acre) cell resolution. The encoding process that helped design the GIS used a stratified random point sampling technique that similar studies had developed and applied (1, 3). The encoding process used a metric grid of 5-kilometer sections, subdivided into 2,500 1-hectare cells (100 meters on a side), aligned with the Universal Transverse Mercator (UTM) Grid System. This grid appears in blue on U.S. Geological Survey (USGS) topographic maps. These maps served as the framework of reference for all data compilation. Within each 100-meter cell, a randomly located point was chosen (see Figure 13) at which the specific factor was encoded as representative for the cell, using a digitizer tablet. This approach allowed extraction of the data from the respective source documents with some rectification necessary for many types of source maps and photographs.

The combination of soil series and cover in each cell helped to calculate the curve number and unit runoff per cell. The 45,000-cell data file was then used to calculate total runoff for a range of events in each of 100 subbasins that averaged 1.95 square miles each. The resultant hydrographs, used in combination with a separate linear data file in GIS describing the hydrographic network of stream geometry, routed and calibrated the hydrologic model (TR-20). NPS mass transport loadings were estimated on an annual basis by cell, again using the land use/land cover data file, and total loads summed by groups of subbasins above critical locations. This issue was particularly important with respect to the drainage areas above the impoundments, where NPS pollutants were of greatest concern.

The soil properties data file was especially useful in evaluating certain management objectives, such as the opportunity for recharging ground-water aquifers. The spatial variation in relative effectiveness of infiltration BMPs was considered for both quantity and quality mitigation because the best methods for NPS reduction usually include recharge where possible. The soil series corresponding with new growth areas were classified regarding their suitability for these BMPs, which are most efficient on well-drained or moderately well-drained soil. Thus, the alternative impacts of future growth could be considered in terms of potential generation (or management) of NPS loads. A BMP selection methodology (see Figure 14), which was developed for the 30 municipalities within the watershed, focused on new land development applications and considered both water quantity and quality management objectives. BMP selection is a function of several factors, including:

- The need for further peak rate reduction.
- The recharge sensitivity of the project site (defined as a function of headwaters stream location, areawide reliance on ground water for water supply, or presence of effluent limited streams).
- The need for priority NPS pollution controls (location within reservoir drainage).

Development of two "performance" levels of BMP selection techniques gave municipalities some degree of flexibility in developing their new stormwater management programs. This system required only the minimally acceptable techniques but recommended the more fully effective ones, hoping that municipalities would strive to incorporate

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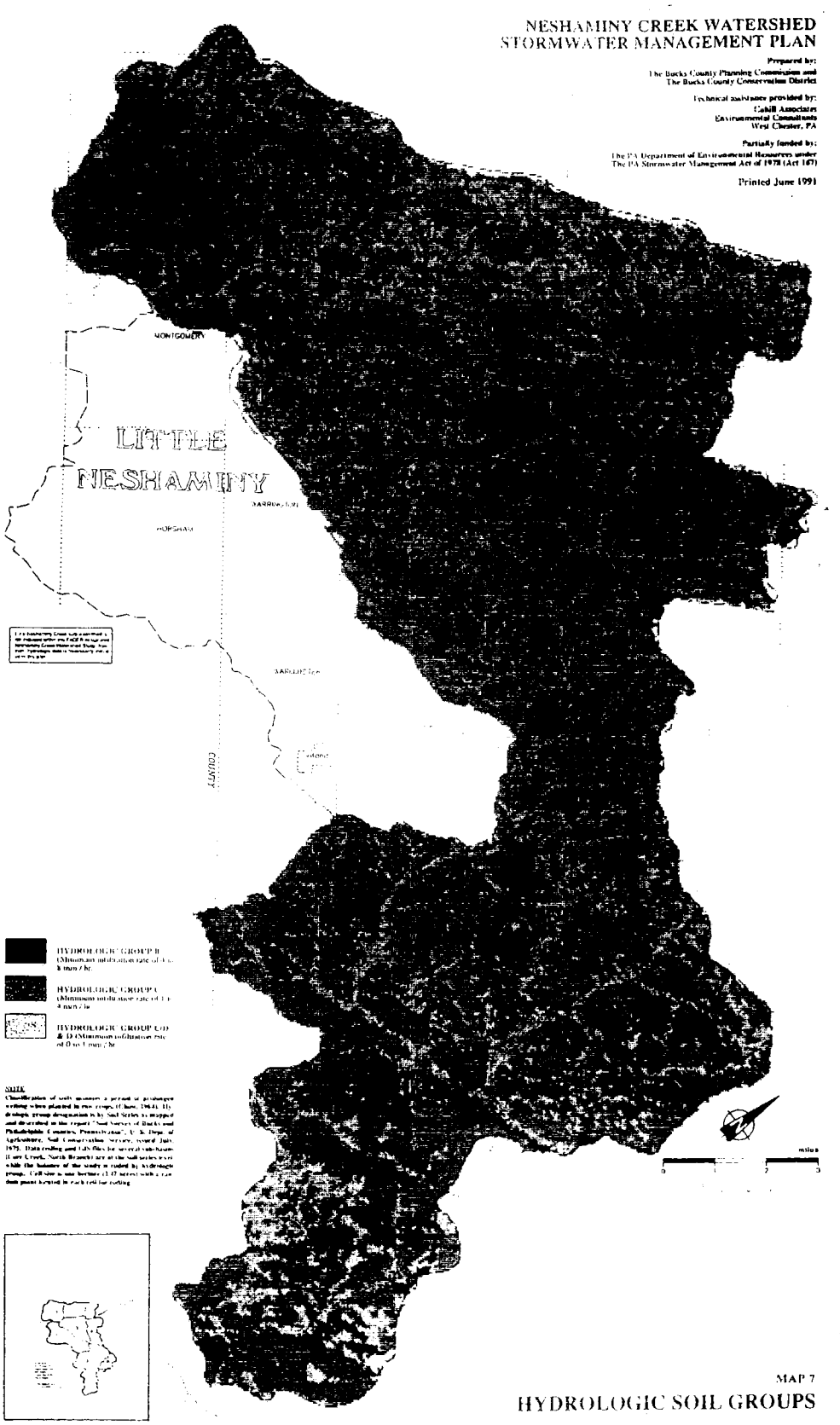

Figure 12. GIS file of hydrologic soil groups in the Neshaminy basin. The 31 soil series are digitized in 45,000 pixels of 1-hectare size.

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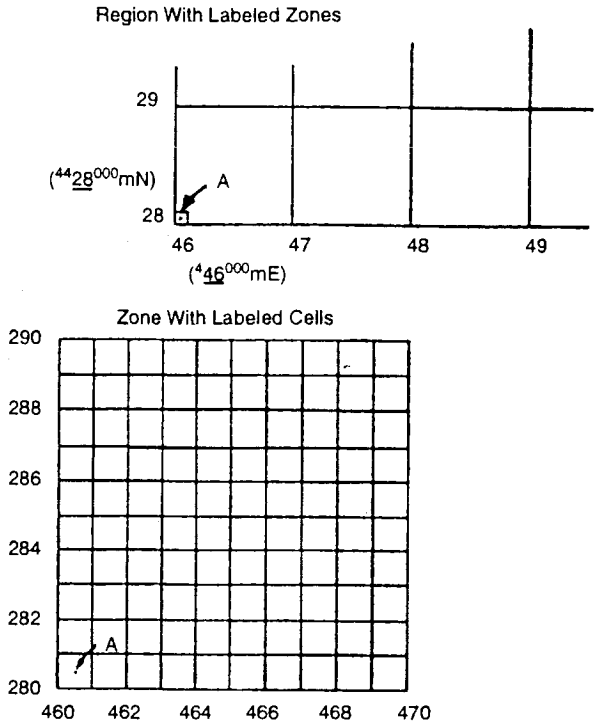

Figure 13. Raster/pixel design of GIS for Neshaminy modeling study. Each pixel is 1 hectare (2.47 acres).

recommended management measures wherever possible. The BMP selection methodology also was sensitive to type of land use or proposed development, assigning typical single-family residential subdivisions different BMPs than, for example, multifamily and other nonresidential proposals (including commercial and industrial proposals). The selection process also determined size of site to be a factor, differentiating between sites of 5 acres or more because of the varying degrees of cost and effectiveness of different BMP approaches. The methodology, if properly and fully implemented, should achieve the necessary stormwater-related objectives—both quantity and quality—that the analysis had deemed necessary (16).

GIS was especially important in its ability to test how reasonable the BMP selection methodology was. Such tests included the ability to evaluate, for each municipality, the following factors:

- The nature and extent of the projected development.
- The size of development/size of site assumptions.
- Other vital BMP feasibility factors such as soils and their appropriateness for different BMP techniques.

GIS also enabled analysis of the water quantity and quality impacts of projected growth on a baseline basis, assuming continuation of existing stormwater management practices. Water quality loadings to individual reservoirs and to the stream system could be readily demonstrated. Because overenrichment of the reservoirs was so crucial, researchers could estimate phosphorus and nitrogen loadings from projected development assuming existing stormwater practices, even on a municipality by municipality basis.

New Jersey Atlantic Coastal Drainage Study

Background

The third study considered a much larger coastal watershed in New Jersey (see Figure 15). The New Jersey

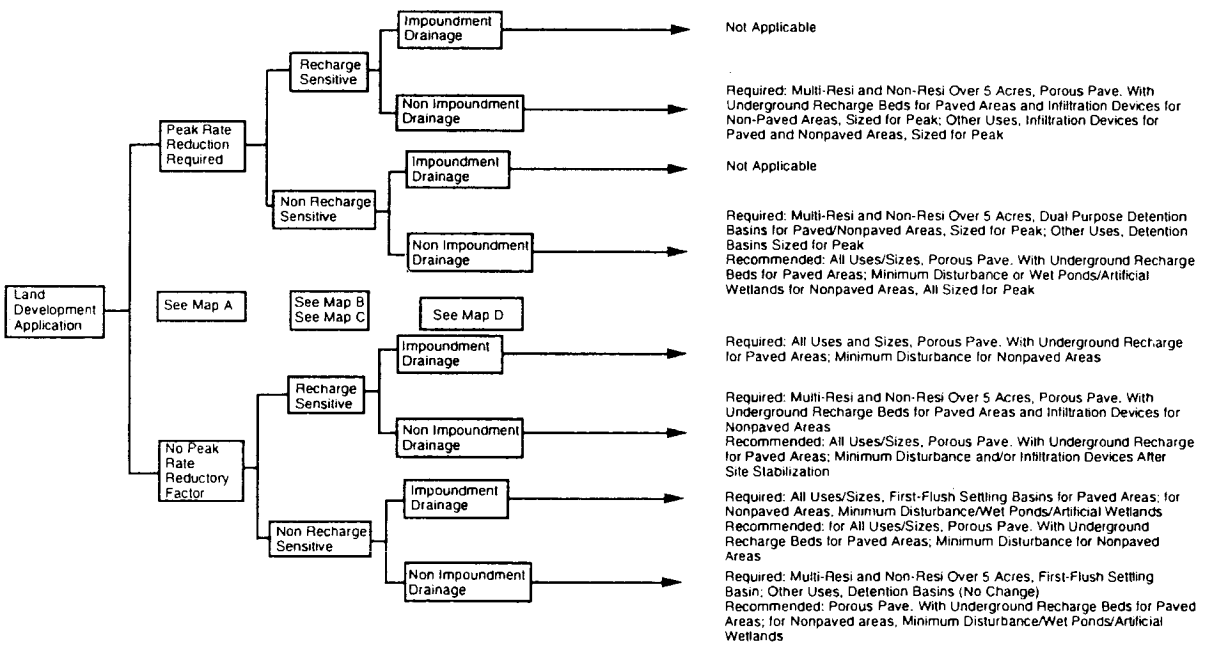

Figure 14. BMP selection methodology used with the GIS database in the Neshaminy basin modeling study.

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Atlantic Coastal Drainage (ACD) includes an area of 2,086 square miles, with barrier islands (50 square miles), wetlands/bays/estuaries (285 square miles), and a unique scrubby pitch pine-cedar forest, known as the Pine Barrens, largely covering the 1,750 square miles of mainland interior (see Figure 16). This flat coastal plain comprises a series of unconsolidated sedimentary deposits of sand, marl, and clay, which increase in thickness toward the coastline. Over the past 16,000 years, as the ocean level has risen, the water's edge has progressed inland to its present position. Ocean currents and upland erosion and déposition have created a long, narrow series of barrier islands that absorb the energy of ocean storms and buffer the estuary habitats from the scour of waves and currents. Between the mainland and barrier islands are embayments and estuaries of different sizes and configurations. Inland erosion and marine sediments have gradually filled many of these areas, creating extensive wetlands (17).

In this ACD region, new land development and population growth have caused significant degradation of water quality from an increase in both point source and NPS pollution. Although the array of pollutants is ominously broad, increased nitrogen and phosphorus loadings have resulted in enrichment of back bays, estuaries, and nearshore waters, contributing to algal blooms, declining finfish and shellfish populations, diminished recreational

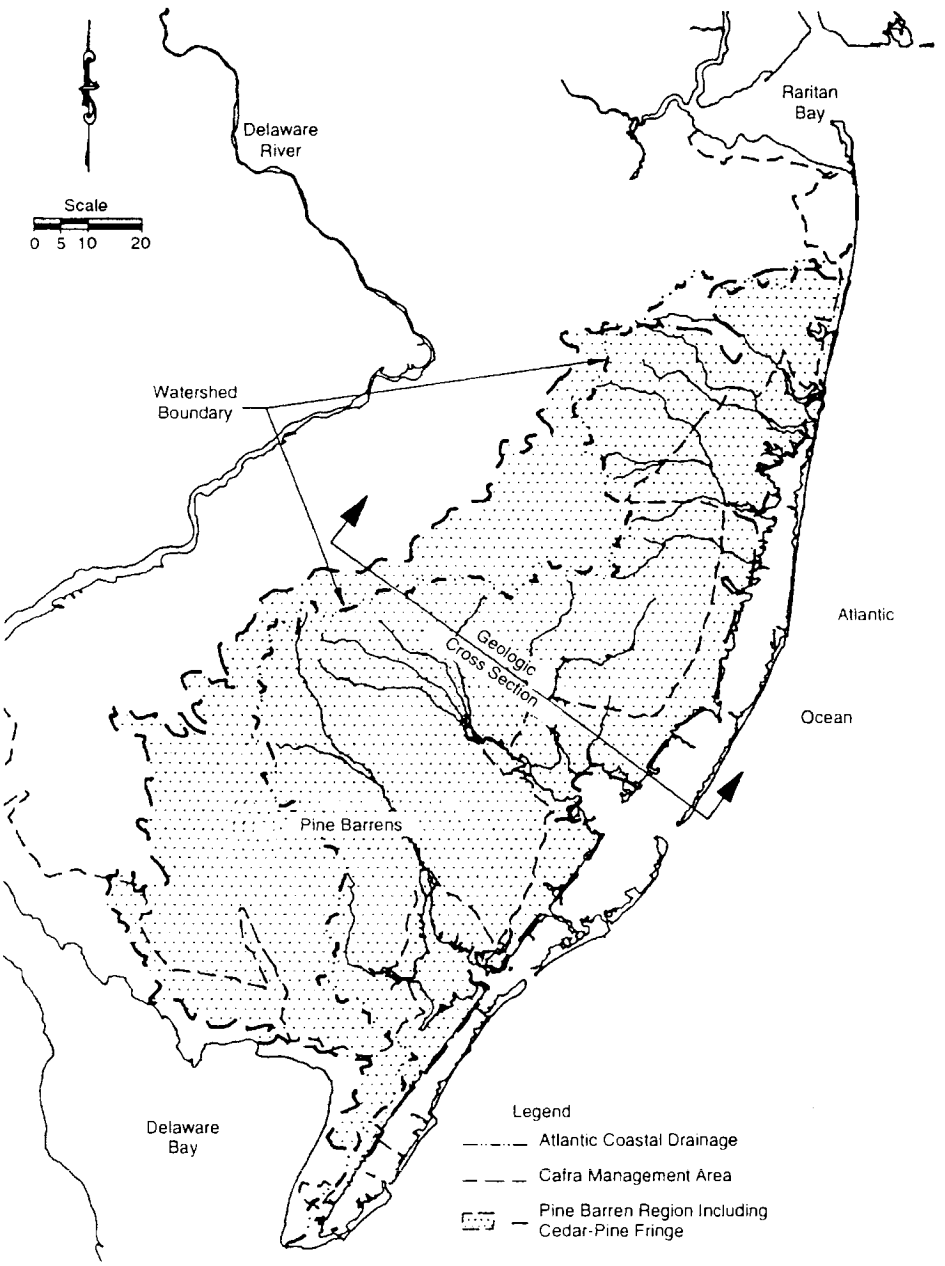

Figure 15. The ACD of New Jersey includes approximately 2,000 square miles of land area from the Manasquan River to Cape May.

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Figure 16. Aerial photograph of New Jersey illustrating the Barrier Islands and estuary system situated along the Atlantic coast.

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opportunities, and a variety of other problems (18). A major source of these nutrients is point source sewage treatment plants (STPs), but the effluent outfalls of almost all these STPs discharge into nearshore ocean waters beyond the barrier islands. Thus, NPS pollutants almost totally dominate the water quality in the estuaries and back bays (19, 20).

These NPS pollutants, which rain scours from the land surface and flushes into coastal waters with each rainfall, comprise a largely unmeasured and unmanaged flux of contaminants. Prior research on coastal water quality has given considerable attention to NPS pollution generated from paved or impervious surfaces, particularly roadways and parking lots where hydrocarbons, metals, suspended solids, biologic oxygen demand (BOD), and other pollutants have been measured.

Although these NPS pollutants are certainly of concern in New Jersey's coastal waters, the enrichment issue has led to a focus on NPS pollution produced when creating large areas of pervious and heavily maintained landscape, such as lawns and other landscaped areas, in the sandy soil context of the coastal area. Typically, significant quantities of fertilizer and other chemicals, which are applied on these new pervious surfaces, are naturally low in nutrients. Although a modest portion of the applied fertilizer runs off directly into surface waters, larger quantities of soluble pollutants, such as nitrates and herbicides, quickly percolate down through the sandy soil, then move rapidly as interflow to the estuary system.

In this coastal drainage of unconsolidated sediments, the hydrologic cycle differs from inland watersheds. Of the 45-inch average annual rainfall, only a small fraction (2.5 inches per year) becomes direct runoff, with the balance rapidly infiltrating into the sand strata (21). Most of the infiltration that reaches the ground water (20 inches per year) discharges to surface streams (17 inches per year) within a few hours following rainfall, producing a lagging and attenuated hydrograph. This rapid infiltration, combined with the sand texture of the soil, has a major bearing on the water quality implications of new land development. Thus, urbanization of coastal regions has dramatically altered hydrologic response, with every square foot of new impervious surface converting what had been approximately 41.5 inches of infiltration into direct runoff to bays and estuaries, with a turbid soup of NPS pollutants.

Even in areas that have maintained infiltration, the coastal soils do not remove NPS pollutants as efficiently as other areas of New Jersey that overlie consolidated formations with heavier clay soils. These soils provide a much more thorough removal of NPS pollutants through physical, chemical, and biologic processes, as rainfall percolates through the soil mantle.

With development of coastal areas, increased impervious areas and changing flow pathways (inlets and storm sewers) convey nonpoint pollutants introduced by development (from both pervious and impervious surfaces) directly to the coastal waters. In addition, freshwater recharge to the underlying aquifer decreases with the increase in impervious surfaces, with resulting increases in saltwater intrusion into the sand aquifers and contamination of ground-water supply wells along the coast. Further compounding the loss of the stormwater for ground-water recharge are increased ground-water withdrawals necessary for new water supply. In sum, urban growth within the ACD, with its 1.13 million permanent residents (and still growing) and an additional 1.5 million summer tourists, has dramatically altered the natural drainage system (and landscape) in a way that significantly increases the discharge of NPS pollutants (22).

GIS Approach

New Jersey's Department of Environmental Protection already had developed a computerized GIS system (ARC/INFO) for environmental analysis and resource planning, so this study aimed to use existing GIS work and to refine this GIS system. Although data files for municipal boundaries, watershed areas, and a variety of other factors already existed, land use/land cover data had not been developed and constituted a major work task. The subsequent land use/land cover file included the entire 2,000 square miles of the ACD, but this focused on the urbanized area (212 square miles) that occupied about 11 percent of the coastal fringe. The end product was a polygon file that described about 2,500 polygons of urban/suburban land, each averaging about 0.1 square miles (see Figure 17).

Using aerial photographs combined with USGS base maps and extensive field reconnaissance, each polygon was classified by:

- Land use type.
- Percentage of impervious cover and maintained areas.
- Degree of maintenance (fertilization) being provided to these maintained areas.

Although classifying land use type and extent of impervious cover/maintained areas was a relatively straightforward evaluation process (rated within one of 11 categories by percentage, 0 to 5 percent, and so forth), the third variable, degree of maintenance, required special treatment and data development procedures. Degree of maintenance was translated into high, medium, and low categories, with high maintenance exemplified by golf courses or other intensively maintained areas. Medium maintenance assumed chemical application rates comparable with those recommended by Rutgers University state agronomists. Finally, low maintenance was typified by a wooded or otherwise naturally vegetated

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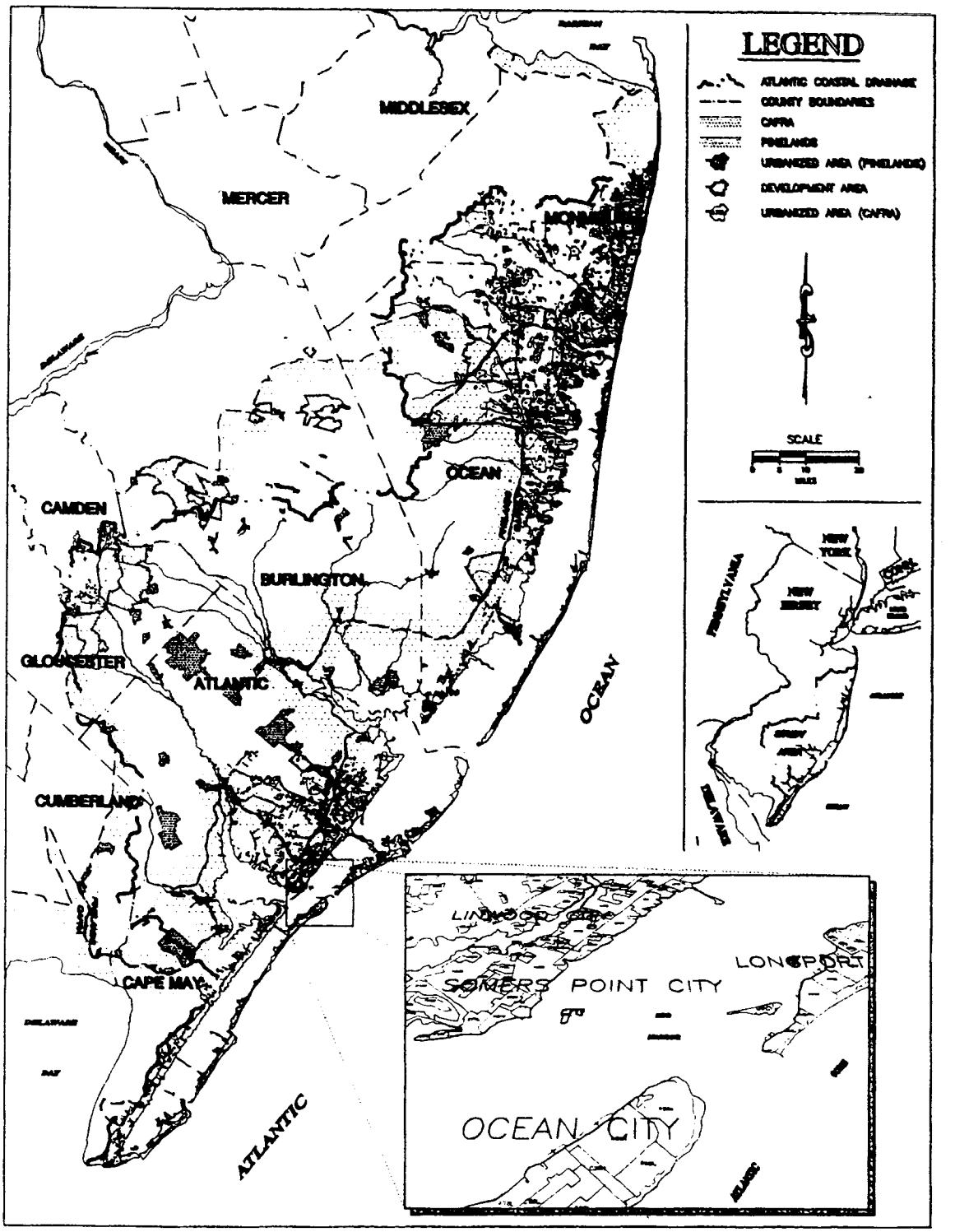

Figure 17. Urban land use polygons digitized for the New Jersey coastal drainage. The 2,500 polygons shown cover approximately 212 square miles (11 percent) of the ACD area of 2,000 square miles.

lot and assumed little or no regular chemical application. Research staff executed considerable field reconnaissance to objectify this judgment-based rating technique (see Figure 18).

Nonpoint Source Analysis

Because the drainage is almost entirely estuarine, the hydrologic aspects in this study were almost irrelevant except as a tool to describe the pollutant transport process. Such coastal drainage systems do not allow the measurement of hydrographs and chemographs (see Figure 6), except on inland riverine segments or selected infrastructure points of discharge (storm sewer outfalls). Thus, the NPS analysis focused on the pollutant production and transport process, especially the nutrients applied to the maintained landscapes, which are a major part of coastal urbanization.

This study required a great deal of effort to produce an index table relating urban cover characteristics (percentage impervious, amount of chemical application) to NPS production potential. For each of the 2,500 urban land polygons GIS described, estimates of the NPS loading for a number of pollutants were generated. Potential loadings were then aggregated by subwatershed. Total NPS loadings could then be compared with point source loadings for the entire coastal drainage. Interestingly, the NPS loading dominated water quality in the estuarine drainage while the point sources, discharged by ocean outfalls to nearshore waters beyond the barrier islands, were the major source of nutrient pollution in this portion of the coastal environment (see Figure 19).

Given the estimates of NPS pollution, the major question involves how to control or reduce these loads. The suitability of selected BMPs for the reduction/prevention of pollutant generation was then evaluated and spatially identified within the drainage (see Figure 20). This figure evaluated the use of constructed wetland systems as a structural measure. That is, GIS allowed state regulators to identify not only what works best in terms of water quality protection measures, but also where these methods could work successfully. This analysis was driven by a detailed evaluation of the combinations of natural conditions GIS identified within the study area. For example, certain BMPs can be applied on soils that have a certain set of characteristics (permeability, depth to seasonal high water table) and that are presently in a given land cover and planned for urbanization.

GIS also aided in evaluating alternative BMP techniques, including reduction in nutrient applications and land management BMPs such as elimination of artificial landscapes, again using a combination of natural features and land use patterns (see Figure 21). The result of this analysis considered the relative proximity of urban land uses to the coastal waters as significantly increasing the potential for NPS transport. State regulatory programs establish minimum setback criteria for development in sensitive areas, and these criteria may be modified to consider pollutant production potential based on GIS delineation of pollutant production.

New Jersey has been striving to develop NPS management programs for coastal areas to reduce existing sources of pollution as well as prevent the creation of

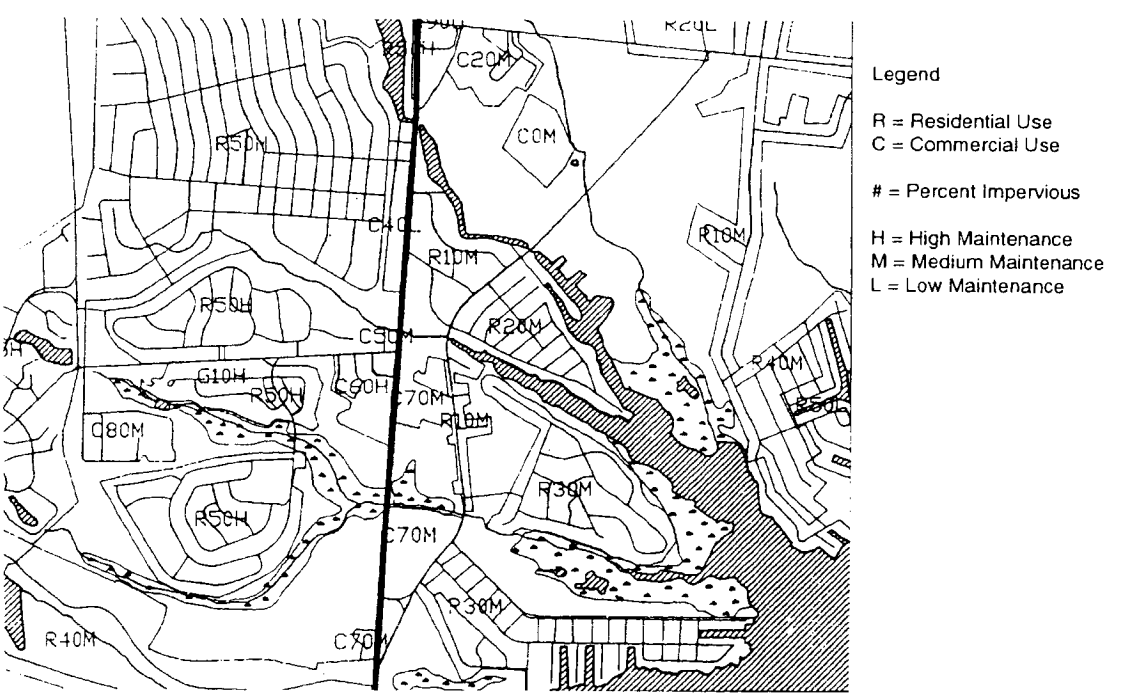

Figure 18. Classification of urban polygons by land use, percentage impervious cover, and degree of land fertilization.

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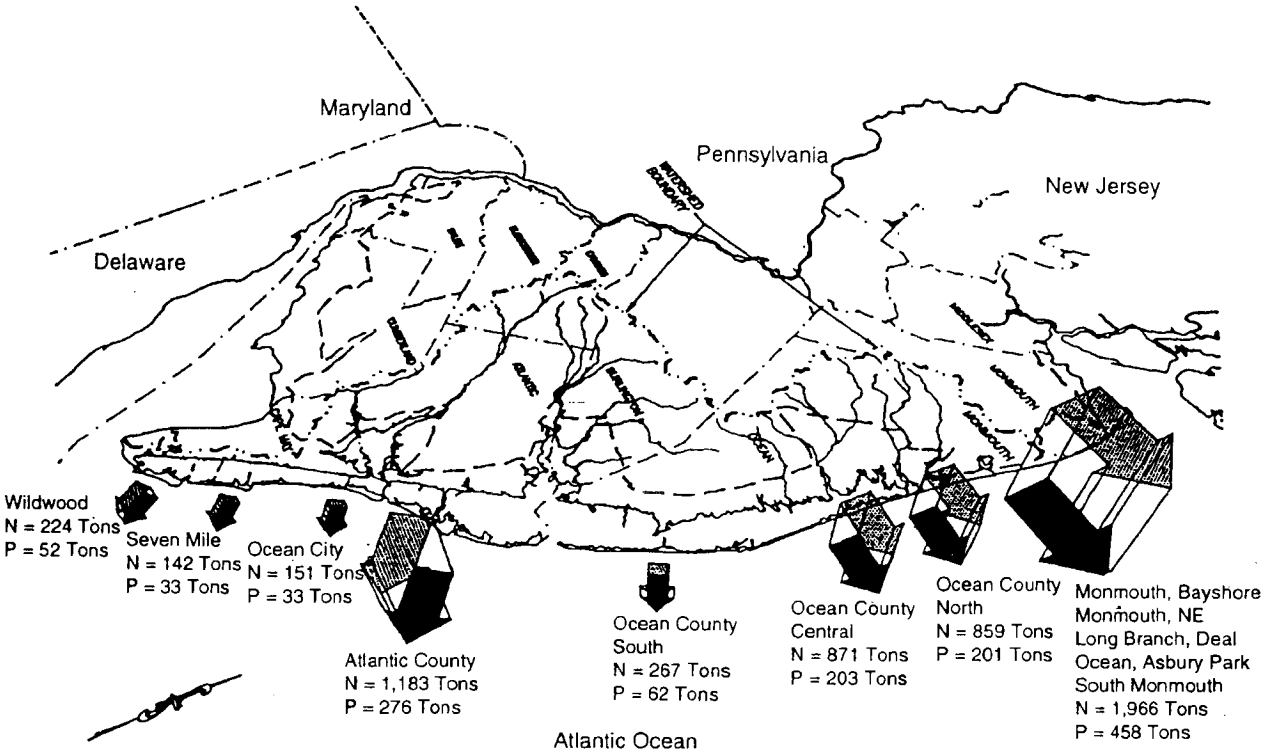

Figure 19. Point and NPS discharges to the ACD. Data shown are in tons of TP and NO_3-N per year.

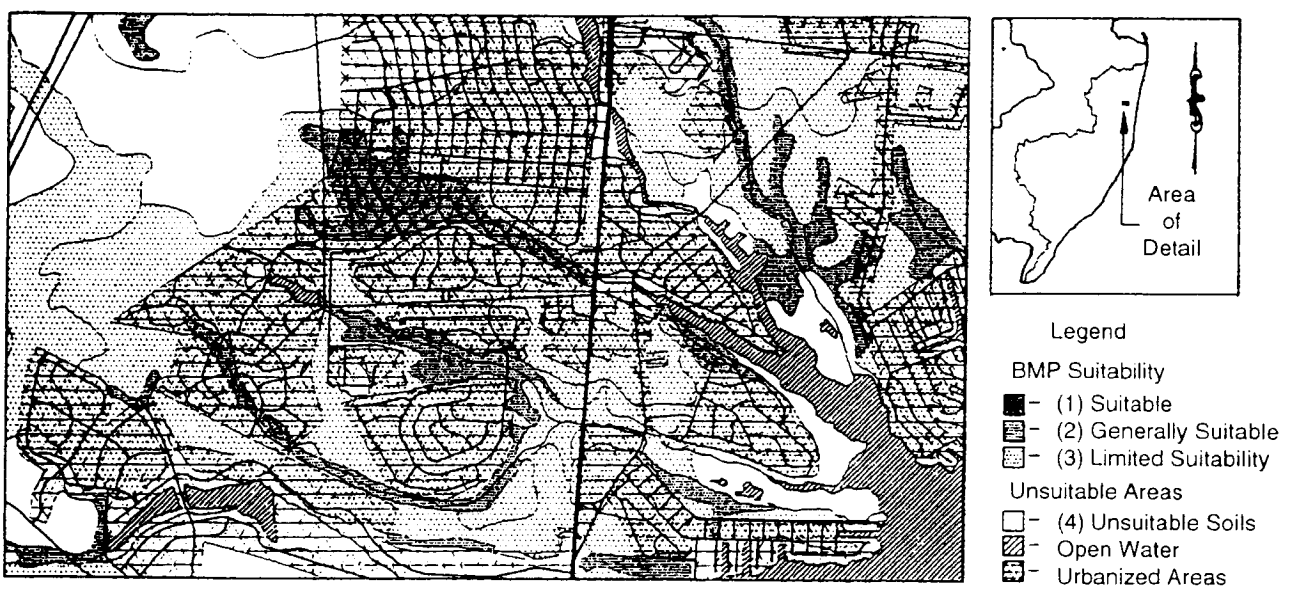

Figure 20. BMP analysis using GIS. Files consider soil suitability, current vegetative cover, and BMP criteria on vacant and developed land parcels.

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pollution. As most regulatory agencies have discovered, NPS management programs can be difficult to implement, especially when confronting issues of land use management. To substantiate the need for new management programs amidst these controversies, the ability to document causal linkages (i.e., to generate data and statistics that make the case for NPS pollutant generators and resultant water quality degradation) is very important. The need for documentation of various types is especially great given the less than perfect data record of water quality in coastal and other waters. All of these factors come together to make the value of a GIS system for water quality management very real.

Conclusion

This GIS-driven analysis indicates that NPS pollutants, especially the nutrients phosphorus and nitrogen, generated from fertilized fields or maintained landscapes surrounding new residential, commercial, and other types of development in drainage systems, contribute significantly to water quality degradation. In effect, the particulate-associated phosphorus and the soluble nitrates serve as surrogates for the full spectrum of NPS pollutants that each rainfall washes from the land. A comprehensive water quality management program must include structural measures to remove pollutants this runoff conveys, as well as management of the contributing landscape to reduce (and perhaps eliminate) the application of these chemicals within the drainage.

In planning new development, management actions should occur on a variety of levels or tiers. On an areawide basis, growth should proceed (with guidance and management) in a manner that would reduce total pollutant discharges; therefore, the total amount of maintained area being created should be as concentrated as possible. On the more site-specific level, measures and construction techniques that reduce the quantity of pollutants generated are essential. Required development guidelines must include, but not be limited to:

- Prevention of excessive site disturbance and ongoing site maintenance (described as a policy of minimum disturbance and minimum maintenance).
- Use of special materials for reduction of stormwater runoff (porous pavement and ground-water recharge).
- Use of stormwater treatment systems (water quality detention basins, artificial wetlands).

In sum, the regulatory framework must contain both "how to build" guidelines, as well as "where not to build" guidelines. GIS can be a powerful tool in both of these processes.

While inland lakes serve as nutrient traps for these NPS pollutants, perhaps the greatest potential impact is the gradual process of excessively enriching our coastal waters. As population continues to migrate to coastal areas, the importance of protecting this fragile ecosystem

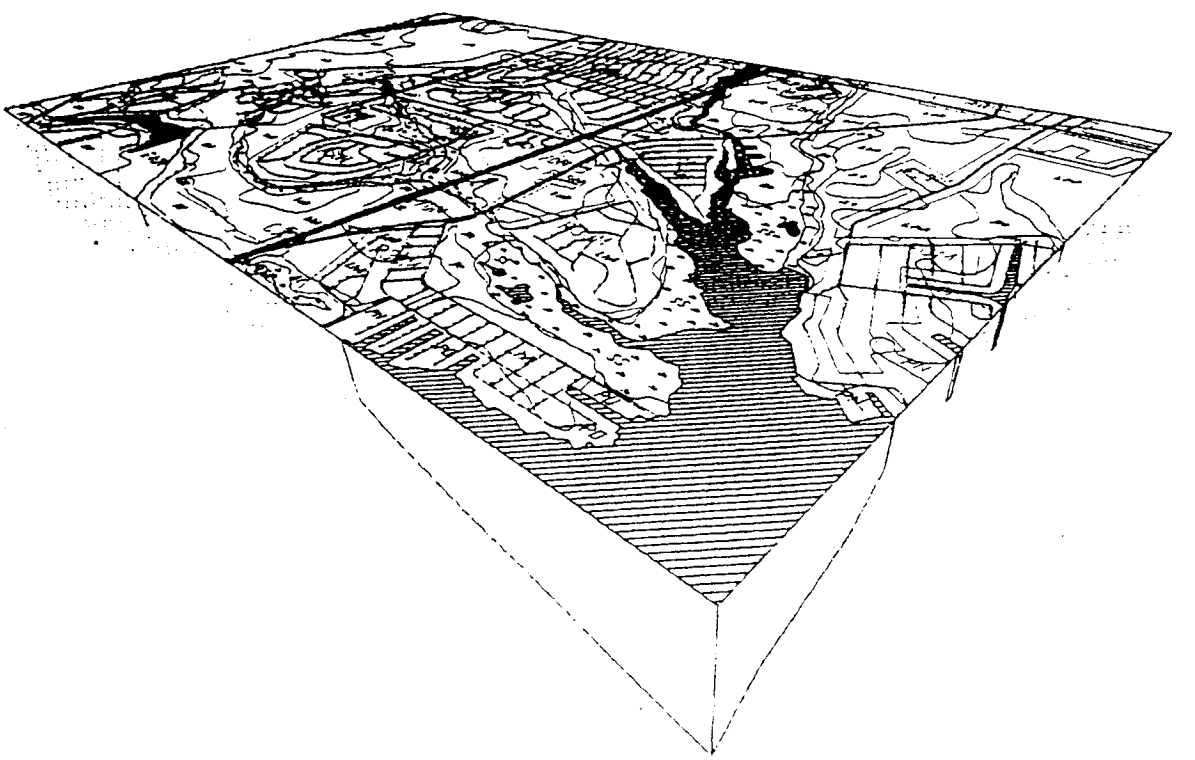

Figure 21. For certain regulatory criteria, the proximity of land uses to the water's edge was a consideration in BMP selection.

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increases. The pollution that new land development generates, including the discharge of point source wastes, should not be allowed to enter coastal waters; it should not be allowed to destroy the natural balance that exists between land and water. The concept of stormwater management takes on an entirely different meaning when viewed as one of the basic mechanisms of this NPS pollution transport. For centuries, engineering of the shoreline has intensively focused on protecting human developments from the ravages of ocean storms. Now, however, the converse seems to be emerging: ocean waters need protection from the impacts of human development.

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Using GIS To Identify Linkages Between Landscapes and Stream Ecosystems

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Introduction

Factors that operate on a variety of temporal and spatial scales influence the structural and functional components of stream ecosystems (1). Quantifying the effects of factors that operate across multiple scales has challenged aquatic scientists over the last several decades. Recently, scientists have recognized that they cannot successfully protect or restore ecosystem integrity without taking into account all appropriate scales; therefore, they are focusing on understanding interactions between terrestrial and aquatic components of entire watersheds (2). Although awareness of the importance of watershed and landscape-scale influences on streams is growing, the tools to examine these influences are still in their infancy.

Most watershed and landscape studies to date have focused on the role watershed-scale parameters play on water chemistry (3-5). These studies usually examined nutrient and sediment inputs from various watershed land covers. Methods for evaluating the patterns in the terrestrial segment of the watershed were awkward and laborious, involving use of planimeters or cutting and weighing maps. More recent watershed studies have attempted to integrate both longitudinal and lateral influences of the terrestrial ecosystems on water quality in streams and wetlands (6-8). This approach takes advantage of newly available tools (geographic information systems and multivariate statistics) for quantifying landscape structure.

Relatively few studies have examined how watershed features influence biological communities. Most studies examining stream biota have concentrated on single land use types (9, 10) or on the relationship between watershed land use and stream physical habitat (11, 12). Typically, study designs have not addressed questions concerning variability of stream communities over relatively large geographic scales.

Our own work centers on identifying linkages between landscape features (watershed scale) and stream reach environments (physical habitat, chemistry), and relating these parameters to major patterns of community variation. In this manner, landscape and reach environment interactions probably control the influence that specific landscape components have on biological communities (13). This is the general premise when using biological communities to assess watershed status. To represent stream biota, we examine benthic macroinvertebrates, which have been used extensively for biomonitoring numerous environmental stresses (14). Macroinvertebrates are sensitive to watershed conditions and exhibit sufficient stability in assemblage structure over time to make them useful as long-term monitors of stream health (15).

This paper presents an overview of our attempts to identify the relative strengths of landscape variables on macroinvertebrate communities. We classify landscape variables into two general categories. The first category, geology and landscape structure (GEOS), considers variables that are fixed on the landscape and are largely uncontrollable by management activities. The second category, land use (LU), includes variables that have anthropogenic origins and may be influenced by land management activities. By understanding the relative strengths these two sets of variables possess in determining community structure, we hope to identify specific species groups that can act as land use and land form indicators. We also hope to identify ways to predict the outcome of specific large-scale land management activities (e.g., silviculture, agriculture) or other large-scale environmental changes (e.g., global warming) on stream ecosystems.

Study Area

This study was conducted in the Saginaw River basin, a 22,562-square-kilometer watershed in east-central Michigan (see Figure 1) that flows into Lake Huron. The Saginaw River watershed was chosen for this study because its component drainages range from heavily affected agricultural to relatively pristine areas.

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Figure 1. The Saginaw basin study area.

Dominating the soils in the lake plain are medium- and fine-textured loams to clays, with sand found in the outwash plains and channels. Artificial drainage and tile systems extensively drain the clay regions. Glacial features such as ground moraines and outwash plains are common. The western sector is characterized by rolling plains with coarse-textured ground moraines. This region contains a high percentage of the forested land, while agricultural land use dominates the eastern sector.

The Saginaw basin covers 16,317 square kilometers, including four major subbasins: the Tittabawassee (6,734 square kilometers), Shiawassee (3,626 square kilometers), Flint (3,108 square kilometers), and Cass (2,331 square kilometers) Rivers. The Tittabawassee subbasin further divides into three principal watercourses—the Chippewa, Pine, and Tittabawassee Rivers. Watersheds adjacent to Lake Huron (Kawkawlin and East basins) are characterized by low topographic relief and elevations averaging 203 and 206 meters, respectively. The Flint and Chippewa/Pine basins average about 278 meters in elevation. These drainages also exhibit the greatest variation in topography.

Study Design

The analysis covers 45 stream sites within the study area. These sites reflect a gradient of land use and physiographic conditions in the Saginaw River drainage. Researchers obtained biological, chemical, and physical samples at one 200-meter stream segment at each site. In addition, a geographic information system (GIS) database was compiled reflecting a number of landscape parameters for the watershed of each stream segment.

Sampling Methods

Macroinvertebrates

At each sampling site, we deployed Hester-Dendy artificial substrate samplers (16) for macroinvertebrate community characterizations twice, in early summer and during base flow conditions in late summer and fall of 1991 or 1992. We allowed samplers to colonize for 6 to 8 weeks. In the laboratory, macroinvertebrates were counted and identified to genus whenever possible. A series of derived variables from the original species abundance tables was used to describe community characteristics. We chose metrics based on their relative utility for examining macroinvertebrate communities, as suggested by Barbour et al. (17) and Karr and Kearns (18). Because macroinvertebrate assemblages are relatively stable through time (15) and preliminary analysis indicated no significant differences between sampling years at stations for which we had 2 years of data (unpublished data), we combined macroinvertebrate data into one database.

Chemistry

We assessed nutrients and other chemical properties related to water quality at each stream site during several periods in the summer and fall of 1991 or 1992. Stream flow during fall sampling was typically less than median flow rates and was considered to represent base flow levels. We used the maximum values of samples taken in June and July to represent summer conditions and the maximum values from September and October to represent fall base flow conditions. The nutrients measured were ammonium (NH_3), nitrate-nitrogen, total nitrogen (TN), orthophosphate (PO_4), and total phosphorus (TP). In addition, we assessed alkalinity (ALK), conductivity, total dissolved solids, and total suspended solids (TSS). Standard methods were used for all measurements (19).

Physical Habitat

We assessed physical habitat during base flow conditions at each stream site in a stream reach that is at least 8 to 12 times the width of the stream segment. A suite of quantitative habitat structure measurements and observations was made at each site. We derived values for six general habitat attributes:

- Substrate characteristics
- Instream cover
- Channel morphology
- Riparian and bank conditions

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- Riffle/Run quality
- Pool quality

Landscape Descriptors

Land use patterns, surficial geology, hydrography, and elevation databases helped to quantify landscape characteristics in the study area (see Table 1). Land use patterns were derived from existing digital data at the Michigan Department of Natural Resources (Michigan Resource Information System [MIRIS] database) (see Table 2). We based classification of land use/cover categories on a modified version of the Anderson (20) scheme, which was constructed specifically for natural resource applications. The result was the following nine land use/cover categories:

- Urban
- Row crop/agriculture
- Other agriculture
- Herbaceous range land
- Shrubby range land
- Nonforested wetlands
- Forested wetlands
- Mixed hardwood forests
- Deciduous forests

In this region, nonrow-crop agriculture is largely represented by pasture, and range lands are predominantly abandoned fields (old fields).

The U.S. Department of Agriculture (USDA) STATSGO soils database enabled the compilation of soil data. The database consists of U.S. Soil Conservation Service (SCS) soil surveys and includes information on dominant texture and drainage in large landscape units. We aggregated soils into simplified categories based on glacial landform.

We delineated watershed boundaries for each sampling station on United States Geological Survey (USGS) topographic maps and digitized them using ARC/INFO (Environmental Systems Research Institute [ESRI], Redlands, California). We identified stream order for each stream segment and coded it as an attribute of the stream reach file. All databases were transformed into a common digital format as necessary and projected into a common coordinate system. We stored data in vector format and analyzed them in ARC/INFO.

Table 2 lists the landscape variables we derived for each watershed. Land use/cover values were reported and analyzed as a percentage of the total watershed area. Patch heterogeneity measured landscape fragmentation and was reported as the number of patches per hectare. We derived slope from elevation data using ARC/INFO. The standard deviation of elevation was used as a surrogate measure of topographic variability.

Statistical Analysis

Using redundancy analysis (RDA), a canonical extension of principal component analysis (PCA), we detected relationships among the individual multivariate data sets. RDA is a form of direct gradient analysis that describes variation in a multivariate data set (e.g., habitat variables or macroinvertebrate metrics) based upon environmental data (21). In RDA, the station scores from a PCA are regressed on a specified set of environmental variables with each iteration, and the fitted values of the regression become new station scores (22). Thus, environmental or predictor variables constrain PCA. Two important outputs from this method are the interset correlations of environmental variables with the RDA axes, which indicate the environmental variables that have the strongest influence in the ordination, and the fraction of

Table 1. Spatial Data Used for Landscape Characterization

Data Layer	Source	Scale	Format Received
Hydrology	U.S. EPA stream reach	1:100,000	ARC/INFO
Elevation	USGS-DEM	1:250,000	Digital elevation model
Land use/cover	USGS	1:100,000	Digital line graph
Land use/cover	MIDNR	1:24,000	Intergraph
Watershed boundary	USGS topographic maps	1:24,000	Manual delineation
Station locality	USGS topographic map	1:24,000	Manual digitizing
Soils	USDA SCS STATSGO	1:250,000	ARC/INFO
Major basin	USGS topographic	1:24,000	Manual digitizing
Quartenary geology	University of Michigan	1:250,000	Manual digitizing

Key: USGS = United States Geological Survey
DEM = Digital elevation model
MIDNR = Michigan Department of Natural Resources
USDA SCS = United States Department of Agriculture Soil Conservation Service

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Table 2. Landscape Variables Measured or Derived for Each Watershed

Variable	Units
Land use/cover	Proportional area
Slope	Degrees
Standard deviation elevation	Meters
Patch heterogeneity	Patches/hectare
Soils	Proportional area
Total area	Hectares

total variance of each predicted variable that is explained by the RDA axes (22).

Performing Monte Carlo permutation tests determined the statistical validity of the association between predictor and predicted variables. Tests were conducted by random permutation of the site numbers in the predictor variables. We randomly linked the predictor data to the predicted data. We conducted 99 simulations to approximate a normal distribution with which to compare our data with random combinations.

We first determined which of the reach variables had strong influences on macroinvertebrate distributions by conducting separate RDAs with physical habitat and chemistry variables as environmental descriptors. We then examined the ability of the landscape data to predict the variation in the important reach variables.

To determine the relative influences of LU and GEOS landscape variables on stream chemistry and physical habitat, we used partial RDA, where one landscape variable type was held constant and variation due to the other landscape set was examined independently. Using this approach, total variation in a multivariate data set can be decomposed in a manner analogous to analysis of variance (23, 24). For this analysis, we attributed variation in the reach variables (habitat and chemistry) to four separate components:

- The variation in reach variables that LU variables explained independently of GEOS variables.
- The variation in reach variables that GEOS variables explained independently of LU variables.
- The variation in reach variables that both GEOS and LU variables shared. This shared variation could have been due to both the dependence of one type of variable on the other as well as noncausal relationships (e.g., the types of soil found in a watershed determine in large part the types of agriculture that can be practiced).
- The variation in reach variables that were unexplainable. This may have been attributable to sampling error, stochastic variation, or other variables not sampled.

Results

Regional Characteristics

Land Use

Land use within the study region was dominated by row-crop agriculture (see Table 3). Individual watersheds ranged from 14 to 99 percent in agricultural land uses, with the East basin watersheds exhibiting the greatest proportion of agricultural land use and the Flint having the lowest proportion of agricultural land use. The Chippewa/Pine and Kawkawlin watersheds exhibited the greatest diversity of land use and cover types within the study region.

Wetlands represented a minor land use component with most watersheds having between 0 and 15 percent land area. The Cass and Kawkawlin basins had the greatest proportion of wetlands, with a median of 6.8 percent for individual watersheds.

Macroinvertebrates

Considerable variation existed among the major basins with respect to the 15 macroinvertebrate community metrics during summer (see Table 4). Metric values for the Flint, Shiawassee, and Chippewa watersheds were similar. Sites within the Kawkawlin and East basins differed considerably from the Flint, Shiawassee, and East basins in several of the metrics. The Kawkawlin watershed was notable for high shredder and filterer proportions and a low proportion of detritivores. The East basin also had a high proportion of shredders. Both the East and Kawkawlin basins had lower proportions of strictly erosional taxa and higher proportions of depositional taxa than the other major basins.

Taxa at the East and Kawkawlin basins also exhibited lower oxygen tolerance than at other major basins. In addition, their Hilsenhoff Biotic Index (HBI) scores (which are sensitive to oxygen availability) were higher than other basins, and they had the lowest EPT (Ephemeroptera, Plecoptera, Trichoptera) richness. Total richness at Kawkawlin was relatively high, however. Richness was highest in the Chippewa/Pine watershed and lowest in the East basin.

In general, fall patterns of macroinvertebrate metrics resembled those of summer. The Kawkawlin and East basins had high HBI scores, low EPT scores, low proportions of erosional taxa, and high proportions of depositional taxa. The proportion of predators was exceptionally high in the Kawkawlin basin due to the abundance and trophic classification of one chironomid genus.

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Table 3. Summary of Landscape Metrics in Six Major Basins of the Saginaw River Drainage

Landscape Variables	East Basin	Cass	Flint	Shiawassee	Chippewa/Pine	Kawkawlin
n	8	7	8	5	15	3
Row crops	86.4 79.7-98.0	58.4 45.1-73.3	38.0 25.6-65.5	43.5 26.7-71.8	48.2 13.9-91.3	26.4 18.5-74.9
Other agricultural land	0.2 0.0-1.2	0.6 0.4-1.4	3.1 0.6-4.1	0.4 0.1-2.0	4.0 0.3-6.6	1.9 0.2-2.3
Urban	0.8 0.1-2.0	1.9 0.4-3.7	8.56 1.4-23.2	10.3 2.4-15.0	2.1 0.8-4.0	1.2 1.0-6.8
Deciduous forest	6.4 0.5-12.1	16.9 5.8-33.4	17.5 10.2-20.5	19.2 16.4-30.4	23.2 3.2-42.1	56.8 13.2-64.5
Mixed hardwood	0.03 0.0-0.3	0.3 0.01-1.7	2.6 0.0-3.8	0.7 0.4-0.9	5.4 0.1-8.5	0.3 0.1-0.3
Range: Herb	1.0 0.1-3.6	6.7 2.1-12.7	13.0 6.6-14.5	12.6 0.9-17.3	6.9 1.0-9.0	2.5 0.7-3.1
Range: Shrub	1.3 0.0-2.3	4.7 3.2-7.7	6.5 2.6-10.5	7.3 2.2-10.1	5.6 0.7-9.0	3.0 2.7-3.2
Forested wetlands	0.1 0.0-0.5	0.1 0.0-0.3	2.3 0.3-3.2	0.7 0.0-3.7	1.0 0.0-2.5	1.8 0.1-2.2
Non-forested wetlands	1.7 0.0-5.3	6.7 1.9-14.5	3.9 0.4-5.2	3.6 0.7-5.0	4.3 0.1-7.9	5.0 1.3-5.9
Slope (degrees)	0.15 0.07	0.29 0.12	0.41 0.11	0.27 0.10	0.35 0.10	0.14 .03
Elevation (meters)	206.5 19.8	239.8 6.7	277.2 22.5	252.6 47.1	278.7 34.9	203.1 9.8
Patch heterogeneity	241.5 102.0	711.4 250.6	950.4 228.6	762.7 223.6	703.6 185.9	519.6 156.7
Watershed area (hectares)	14,968.9 11,132.0	38,117.7 58,321.9	28,926.1 19,236.7	46,530.0 50,798.0	53,704.2 44,872.8	22,240.2 3,848.0

Land use/cover variables (agricultural land through nonforested wetlands) are reported as median and range; landscape structure variables (slope through watershed area) are reported as mean and standard deviation. Land use/cover represents proportional areas of each watershed.

Identification of Important Reach-Scale Variables

Chemistry

RDA showed that chemical variables explained 26 percent of the variation in macroinvertebrate data in summer and 33 percent in fall. The most important variables in summer were TN and TSS (see Table 5). Fall macroinvertebrate communities were influenced by a greater number of variables, including NH_3, TP, ALK, and TSS.

Physical Habitat

The 13 physical habitat variables explained 37 percent of the macroinvertebrate data in summer and 46 percent of the macroinvertebrate data in fall. In summer, the percentage of deep pools and canopy extent along with channel dimensions, such as bank full width (BFW) and bank full depth (BFD), were the most important variables. In fall, the percentage of fines and deep pools as well as canopy extent were among the most important variables (see Table 6).

Landscape Influences on Surface Water Chemistry

In summer, the landscape data explained 55 percent of the variation in chemical variables. The proportion attributable to LU was larger than that attributable to GEOS (see Figure 2). The two data types shared 12 percent of

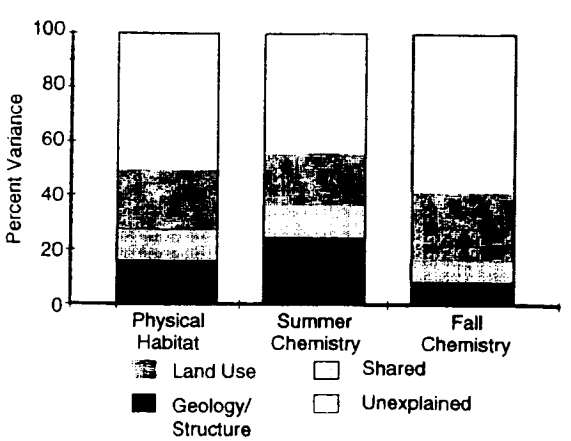

Figure 2. Results of variance decomposition from partial RDA.

Table 4. Mean and Standard Deviation of Macroinvertebrate Metrics Calculated for Summer Collection Periods for Six Major Basins of the Saginaw River Drainage

	East Basin	Cass	Flint	Shiawassee	Chippewa/Pine	Kawkawlin
n	8	7	8	5	15	3
Chironomidae	59.1 35.3	57.9 20.3	45.9 27.9	32.2 28.5	45.5 29.6	67.1 18.6
Omnivores	19.4 13.7	19.1 7.1	18.1 13.9	14.4 3.8	21.5 9.8	22.0 16.7
Detritivores	57.1 34.1	69.7 9.7	75.3 16.0	79.9 6.4	70.4 9.8	29.0 26.1
Shredders	30.3 33.9	18.7 5.1	10.6 6.0	7.7 6.8	14.4 15.6	51.0 26.9
Gatherers	59.8 32.9	18.7 5.1	64.3 12.8	65.0 14.1	65.8 17.2	39.8 30.1
Filterers	27.4 38.1	23.4 18.5	22.4 14.1	18.9 12.8	17.6 15.1	39.9 35.7
Grazers	32.2 34.4	13.6 16.2	26.2 21.0	40.1 22.7	25.5 21.1	25.4 22.9
Predators	1.5 2.2	1.2 1.2	1.5 2.0	1.0 1.0	1.4 0.8	1.9 0.8
2 Dominants	64.5 25.5	54.3 6.1	50.3 15.2	54.2 9.5	51.9 11.6	60.0 21.7
Total abundance	2077 4951	650 739	574 622	325 91	497 230	433 297
HBI	7.1 1.4	5.6 ?	5.6 0.8	6.0 0.5	5.1 1.1	8.1 0.8
Erosional taxa	25.9 12.4	36.1 5.5	35.5 9.5	38.9 14.7	36.1 11.0	14.9 5.3
Depositional taxa	35.5 13.2	23.7 9.6	27.5 11.5	27.0 6.6	25.4 10.7	52.4 6.5
Species richness	17.2 4.5	18.3 9.6	22.1 8.2	20.6 4.7	26.6 3.0	23.3 4.9
EPT taxa richness	5.0 2.7	5.7 2.8	7.3 3.0	8.0 2.3	10.0 3.7	3.3 0.5

the variation. The relationship between LU variables and chemistry was significant ($p < 0.05$), and the relationship between GEOS variables and chemistry was not significant ($p > 0.05$).

In fall, variation explained by LU was proportionally less than during summer (see Figure 2). GEOS landscape variables explained 25 percent of the total variation while LU variables accounted for less than 10 percent of the total variation. LU and GEOS variables shared approximately 8 percent of the variation. In contrast with summer, GEOS variables were significant and LU variables were not significant when examined with the Monte Carlo test ($p < 0.05$).

The importance of GEOS and LU variables in explaining variation in the chemistry variables, as well as the total amount of variation explained, differed considerably among the chemistry variables. Figure 3 shows only summer data. For example, the landscape variables explained almost 80 percent of the total variation in TN (see Figure 3). The largest proportion of variance was explained by the shared influences of GEOS and LU. Alkalinity, which was also well predicted, however, was much more influenced by variation attributable to LU variables. In comparison, LU variables explained less than 45 percent of the total variance in TP, and the majority of this variance was attributable to GEOS.

To further examine the influence of specific landscape variables, we compared the various axes in the significant partial ordinations (see Figure 4). In summer, when we observed a significant effect of LU variables on water chemistry, forested land covers and nonrow-crop agriculture had their greatest influence on TSS and ALK. LU heterogeneity and shrub vegetative cover most strongly influenced both TN and NH_3. In fall, when GEOS variables had a significant relationship to water chemistry, ALK and NH_3 were more influenced by peat land soils and watershed size. The proportion of sand and gravel soils, as well as clays, explained much of the variation in TN and TSS.

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Table 5. Chemistry Variables That Had a Correlation (r) of at Least 0.30 With One of the RDA Axes With the Summer or Fall Ordinations; a Monte Carlo Analysis Indicated That Both Summer and Fall Ordinations Were Significant ($p < 0.05$)

	RDA 1		RDA 2		RDA 3	
Variable	Summer	Fall	Summer	Fall	Summer	Fall
TN	0.44	0.03	-0.04	-0.07	0.05	-0.14
NH_3	0.05	0.04	0.09	0.52	0.07	-0.09
TP	0.12	0.40	0.07	0.26	0.25	0.14
PO_4	-0.04	0.19	0.01	-0.08	0.26	0.34
TSS	0.01	-0.31	0.43	0.36	0.22	-0.13
ALK	-0.08	-0.36	-0.16	0.09	0.15	0.2

Table 6. Physical Habitat Variables That Had Correlations (r) Over 0.3 With the Ordination Axes; Results of the Monte Carlo Simulation Indicated That the Fall but Not the Summer Ordinations Were Significant ($p < 0.05$)

	RDA 1		RDA 2		RDA 3	
Variable	Summer	Fall	Summer	Fall	Summer	Fall
Percentage of fines	0.27	0.47	0.04	0.28	0.18	-0.09
Percentage of shallows	-0.06	0.17	-0.28	-0.11	0.32	-0.38
Wood	0.09	0.34	-0.1	0.02	0.10	-0.17
Percentage of deep pools	0.50	0.54	0.16	0.09	-0.17	0.26
Erosion	0.16	0.30	-0.3	-0.31	0.01	0.27
Maximum depth	0.08	-0.14	0.08	0.16	-0.39	0.13
Canopy extent	0.75	0.21	-0.47	-0.42	0.21	-0.25
BFW	-0.10	-0.23	0.52	0.36	0.07	0.03
BFD	-0.09	-0.13	0.32	0.02	-0.08	0.22
Flood ratio	0.03	-0.30	0.07	0.23	-0.41	0.02

Landscape Influences on Physical Habitat

GEOS landscape variables attributed for the largest portion (22 percent) of the explained variation in physical habitat variables (see Figure 2). LU variables accounted for 16 percent of the explained variance. The partial ordination for GEOS but not LU was significant as the Monte Carlo procedure determined.

As noted with chemistry variables, there were considerable differences in the ability of the landscape variables to predict individual habitat characteristics. Landscape variables were best at predicting BFW and least powerful for predicting the percentage of deep pools (see Figure 5). BFW was influenced predominantly by GEOS and only minimally by LU. The most influential GEOS variables for BFW related to watershed area (see Figure 6). Woody debris was predominantly influenced by LU variables. The most influential LU variables for woody debris related to forested wetlands. Flood ratio was intermediate to these examples. Both sets of landscape variables shared the largest proportion of explained variance for this parameter.

Discussion

Our studies demonstrate the distinct influences landscape features have on stream macroinvertebrate communities through modifying surface water chemistry and stream habitat. Land use most strongly influences stream chemistry during summer months when surface runoff and soil leaching are greatest. In addition, fertilizer application in row-crop agriculture is highest in the first part of the growing season. The strong relationship between some aspects of land use and stream water chemistry were similar to those observed in other studies (4, 6, 7). The specific mechanism by which stream chemistry influences macroinvertebrates is not clear. The addition of nutrients can significantly affect stream productivity (25-27); however, light often limits primary production in agricultural areas (28-30). Nutrients may

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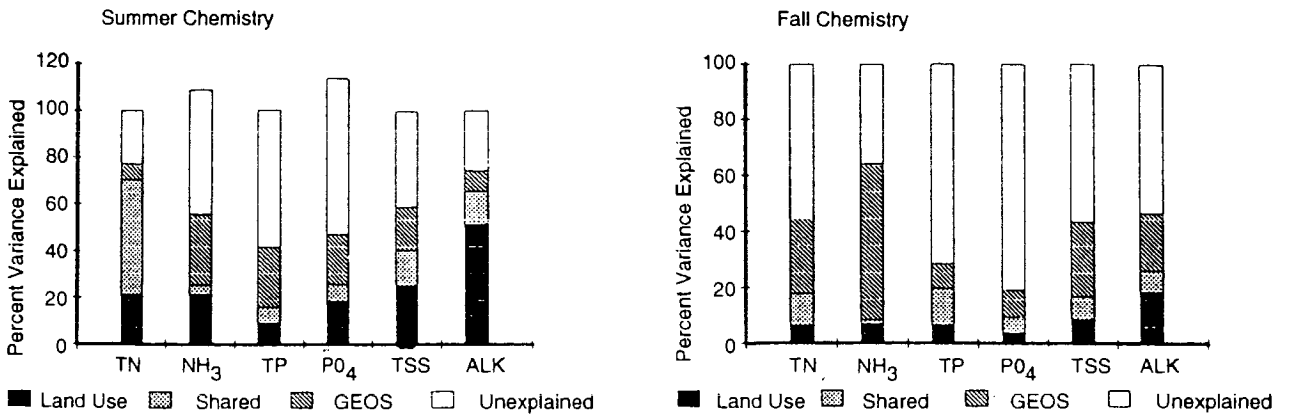

Figure 3. Results of variance decomposition for chemical parameters from partial RDA.

also act as indicators of other agricultural chemicals (e.g., herbicides, pesticides) and carbon sources, which we did not measure in our study, but their inputs are moderated by agricultural land use.

The influence of landscape features on stream habitat may be even more complex than stream chemistry can show. Low-gradient midwestern streams in the United States are under a complex set of controls related not only to subtle topographic gradients but also to interactions among land use, soil type, riparian vegetation, and historic anthropogenic modifications. Ditching, channelization, and hydrologic changes associated with large-scale land-cover changes, such as logging and wetland drainage, have a significant effect on current stream physical conditions.

Our analysis did not quantify many of these effects. Nonetheless, some land use influences seem clear. First, we found that the extent of permanent vegetation in a watershed played a central role in land use impacts on stream habitats. All of the key factors that described variation in the habitat variables included land covers with permanent vegetation. In addition, our initial land-cover analysis found that row-crop agriculture highly negatively correlated with almost all permanently vegetated land covers. Second, although wetlands represented a relatively small proportion of total watershed area (less than 5 percent), they had a relatively high influence on habitat features. Both forested and non-forested wetlands were important to habitat features such as woody debris and some aspects of channel dimensions.

We found that one of the most important landscape influences on stream habitat was watershed area, which explained much of the variation in channel dimensions. These habitat parameters, in turn, were among the most important for explaining variation among macroinvertebrate communities. Stream size, including channel dimension, is an important determinant of several macroinvertebrate community characteristics that stem from longitudinal phenomena associated with stream ecosystems (31, 32). Because these relationships are fixed, they are not amenable to manipulation by management. Consequently, if the monitoring objective is to

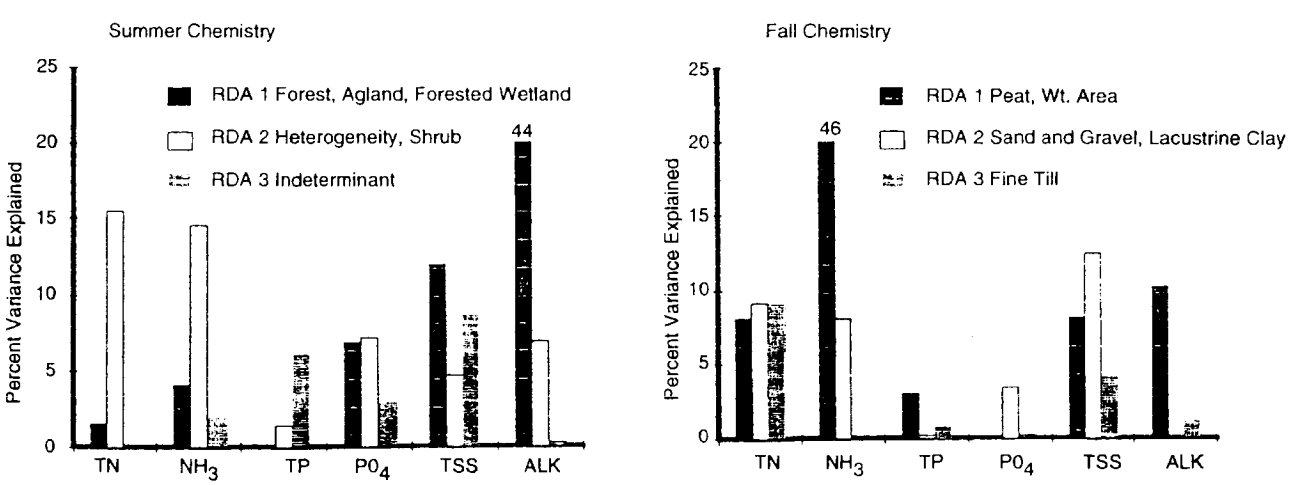

Figure 4. Factors influencing chemical parameters in summer (land use) and fall (geology/structure).

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detect changes resulting from land use, then specific macroinvertebrate community characteristics that respond to changes in channel dimensions are not good biological indicators.

Our study indicates that land use and geology/structure have similar magnitudes of influence on reach environments and biotic communities. This suggests that although some community and water quality characteristics are controlled by landscape features that are difficult to modify or manage, other aspects may be altered through specific activities designed to increase or decrease land uses for a desired effect.

The methods described in this study show promise for examining landscape-stream interactions. Much of the variation we found in both stream reach environments and macroinvertebrate communities, however, remained unexplained. Undoubtedly, we can attribute much of this unexplained variation to the abbreviated sampling procedures that large-scale field studies such as this dictate. Questions of scale and position, however, may also be responsible for much unexplained variation.

The minimum mapping unit used in our study (1 hectare) may be insufficient to detect local land use practices that influence streams. For example, riparian zones along many streams may only be a few meters in width and yet exert considerable control over nutrient inputs (33). Higher spatial resolution may improve predictive power. In addition, the position of certain landscape elements may determine consequent effects on stream conditions. Johnston et al. (7) found that wetland position within a watershed influenced the ability of the wetland to modify stream chemistry. With respect to biologic communities, the abundance of some taxa may relate to the presence of upstream refugia, which can provide sources of colonists (34) and other biogeographic effects.

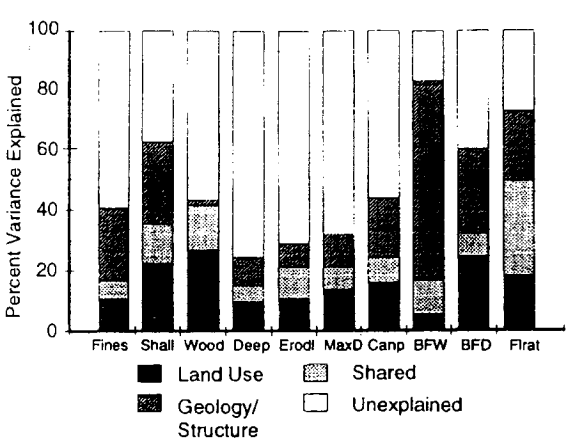

Figure 5. Results of variance decomposition from partial RDA for physical habitat.

The further development of empirical relationships between landscape parameters and streams will provide important insights into understanding the spatial and temporal scales needed for modeling and monitoring large watersheds. Government and commercial geographic databases that can be easily adapted to GIS technology should become more readily available. As they do, the ability to identify relationships across scales, and consequently predict stream community composition from watershed-scale attributes, will facilitate the assessment of ecological risk associated with land use changes.

Acknowledgments

Jim Wessman, Frank Kutka, and Tim Aunan developed much of the data reported in this study. John Arthur and Tom Roush collected most of the chemistry and macroinvertebrate data and provided invaluable logistical support. Cajo ter Braak provided helpful insights to statistical analysis. This research was funded by the U.S. Environmental Protection Agency through CR-818373-01-0.

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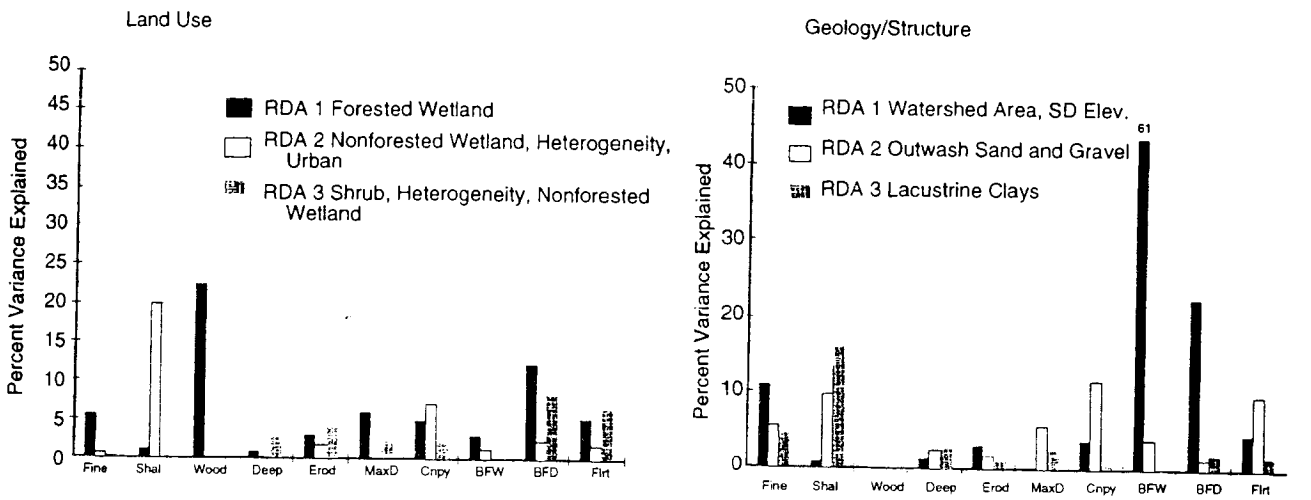

Figure 6. Factors influencing physical habitat parameters.

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Nonpoint Source Water Quality Impacts in an Urbanizing Watershed

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Abstract

As part of the larger Narragansett Bay Estuary Project, the University of Massachusetts Cooperative Extension Service contracted with the university's METLAND research team to develop a geographic information system (GIS) database, generate watershed-wide maps, perform analyses, and develop a modeling procedure. The objective was to educate local officials about the impacts of development on water quality and to help local boards minimize the effect of nonpoint sources of pollution.

Because the receiving waters of the Narragansett Bay are located far downstream in Rhode Island, the upstream communities in Massachusetts are reluctant to enact measures to improve water resources outside of their jurisdictions. A GIS was used to create awareness of existing downstream problems and to show the upstream communities how development will ultimately affect water resources in their own backyards.

To nurture this awareness, a "buildout" analysis was conducted for an entire upstream subwatershed, the Mumford River watershed, containing parts of four towns, and roughly 50 square miles. This buildout was coupled with a loading model using Schueler's Simple Method to illustrate the potential impacts of future development, and encourage local boards to minimize future nonpoint sources of pollution.

GIS proved its usefulness by developing customized maps for each town, by generating several "what if" scenarios showing the impacts of different zoning changes, by facilitating long-range planning for small towns without professional staff, and by encouraging a regional perspective on development issues. The entire planning process was most successful in creating a series of partnerships that will continue after the grant expires. The university shared coverages with the state GIS agency, creating new coverages not previously available, specifically soils, ownership, and zoning. Small towns learned about the potential of the new technology. Students gained from hands-on experience with real-world problems. State agencies saw their efforts understood at the local level, especially as they reorganize on a basin approach and begin to implement a total mass daily loading (TMDL) procedure to coordinate permitted discharges and withdrawals.

As greater emphasis is placed on controlling nonpoint sources of pollution, more attention needs to be focused on local boards, who control land use decisions in New England.

Introduction

Project Description

Narragansett Bay is a vital resource for southern New England. The health of its waters is critical to the regional economy, supporting fisheries, tourism, and quality of life. Increased development along the bay's shorelines and throughout its drainage basin threatens the quality of these waters, however. The U.S. Environmental Protection Agency (EPA) recognized the threats to this important water body and designated the Narragansett Bay under its National Estuary Program in 1985.

Completing a Comprehensive Conservation and Management Plan (CCMP) for Narragansett Bay took 7 years. The CCMP identified seven priority areas for source reduction or control, including the reduction of agricultural and other nonpoint sources of pollution. The nonpoint source strategy identified United States Department of Agriculture (USDA) agencies, conservation districts, and other public and private organizations as having principal roles in nonpoint source management.

Whereas the vast majority of Narragansett Bay lies within the boundaries of Rhode Island, a significant portion of its pollution load originates in Massachusetts. Recognizing that the watershed extends beyond state boundaries, the USDA provided 3 years of funding to Cooperative Extension and the Soil Conservation Service (SCS) in both Massachusetts and Rhode Island to

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coordinate their efforts in an innovative attempt to reduce the impact of nonpoint sources of pollution on Narragansett Bay. While water quality is a relatively new focus for Cooperative Extension, it fits well with the historic mission of extending the knowledge base of the land-grant colleges out into the community, and providing training and capacity building for local officials and community organizations.

With such a large area of concern, the management team decided to focus on a smaller subwatershed area in each state for the first 2 years. The strategy was first to develop a program for the mitigation of nonpoint source pollution on the smaller scale of a watershed of roughly 50 square miles, then take the lessons learned and apply the most appropriate efforts throughout the larger watershed. By using similar strategies in Rhode Island and Massachusetts, but choosing subwatersheds that differ in terms of location relative to the receiving water, size, staffing, and sophistication, the two states gained from each other's experience, sharing the successful techniques and avoiding each other's mistakes.

For its pilot study, Rhode Island chose Aquidneck Island, home of Newport, Portsmouth, and Middletown, with a special focus on protecting surface water supply reservoirs. Massachusetts chose an upstream watershed in the Blackstone Valley, somewhat rural in character, but rapidly undergoing a transformation to suburbia.

Watershed Description

The Blackstone River drops 451 feet in its 48-mile journey from Worcester, Massachusetts, to Pawtucket, Rhode Island. In the 19th century, this drop of roughly 10 feet per mile was ideally suited to providing power to mills during the early years of the industrial revolution. By the Civil War, every available mill site was developed, earning the Blackstone River the name "The Hardest Working River."

The Blackstone has a long history of pollution. First, the textile industry, then steel, wire, and metal finishing industries used the river for power, in their manufacturing process, and for waste disposal.

In Massachusetts, the Blackstone River is the major source of many pollutants to Narragansett Bay. Based on total precipitation event loading calculations, the Blackstone River is the principal source of solids, cadmium, copper, lead, nitrate, orthophosphate, and PCBs to the bay (1). The Blackstone River has an average flow of 577 million gallons per day or 23.2 percent of the freshwater input to the bay.

The watershed area in Massachusetts equals 335 square miles; with a population of 255,682, this results in a density of 763 people per square mile. The Blackstone Valley has 9,000 acres in agricultural use, with more land in hay (4,500 acres) than crops (3,700 acres) to support its 4,400 animals.

Based on aerial mapping flown in 1987, the Blackstone Valley has lost 5 percent of its cropland, 9 percent of its pasture, and 21 percent of its orchards since 1971. The valley remains more than 60 percent forested, but that represents a decrease of 5 percent. The forest and farmlands were lost to development as low density housing grew by 45 percent, commercial use grew by 15 percent, and transportation grew by 54 percent. Waste disposal grew 52 percent to 582 acres, and mining, which in this region represents gravel pits, grew 22 percent to 1,100 acres.

Watershed soils consist mainly of compact glacial till on rolling topography, with 3 to 15 percent slopes. The river and stream valleys are underlain by glacial-derived sand and gravel outwash, which provide drinking water to all towns in the area except Worcester and support the large gravel pits. The high clay content in the till soils of the uplands makes for a high water table, which is beneficial for growing corn but causes problems for septic systems.

Following a preliminary study of the subwatersheds, the Mumford River in the Blackstone Valley was selected as the focus watershed based on its size, location, land use, and existing water quality (see Figure 1). The Mumford River watershed has an area of 57 square miles, with a length of 13 miles, and lies within the towns of Douglas, Northbridge, Sutton, and Uxbridge. These towns share the attributes of small, rural communities undergoing rapid development, with no professional planning staff (see Figure 2). According to the 1990 Census, Douglas grew 46 percent in 10 years to 5,438; Uxbridge experienced 24 percent growth to 10,415; Sutton increased 17 percent to 6,824; and Northbridge grew 9 percent to 13,371.

Project Strategy

Because the generation of nonpoint sources of pollution is so closely tied to land use, and because local boards composed of citizen volunteers have principal control over land use in New England, the key focus of this program is to train local boards to recognize and begin managing the threat that nonpoint sources of pollution pose to water quality. Local planning boards, conservation commissions, and boards of health address land use issues and can regulate and shape existing and proposed development. By developing a program to train local officials, Cooperative Extension can focus its outreach where it will have the greatest impact in both the short and long term. Local boards have the strongest opportunity to comment on how land is to be used as it undergoes development. Therefore, this project focused on preventing future deterioration as opposed to fixing

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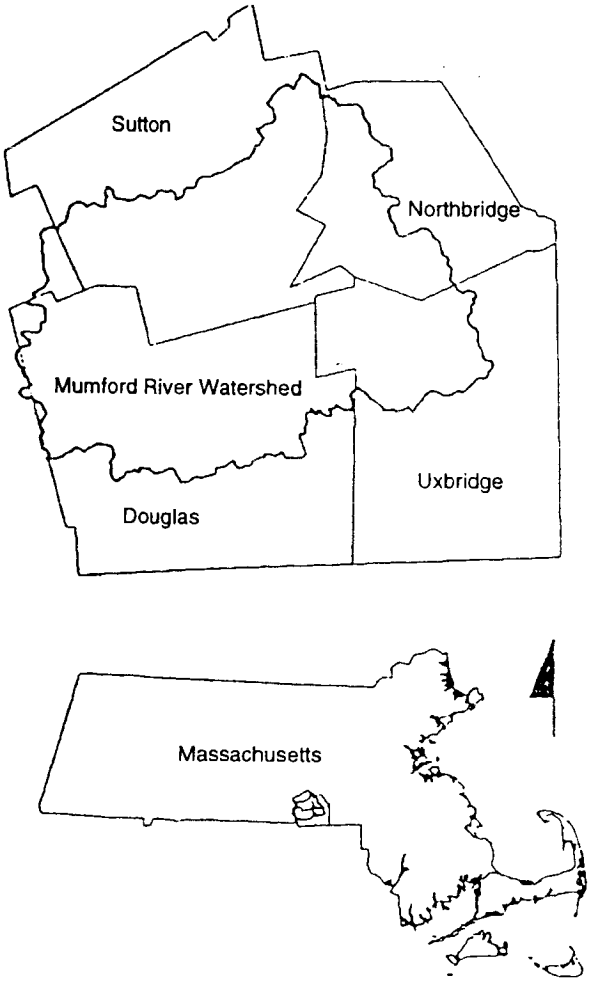

Figure 1. Map of Mumford River watershed study area.

existing problems. This is especially appropriate in a rapidly urbanizing setting.

Both Massachusetts and Rhode Island chose to utilize GIS technology because of its ability to store, analyze, transform, and display geographic, or spatial information. Its database management and analytical capabilities make it a useful tool for pollution load modeling and buildout scenario development, while its mapping capabilities make it an excellent tool for sharing information with local officials. This paper documents a case study on how GIS technology was used to apply a watershed-wide pollution loading model and to develop buildout scenarios for demonstrating to local officials the potential impacts of future development on water quality.

This project used GIS in four different applications:

- *Printing customized, large-scale maps:* This most basic application of a GIS proved the most useful for local officials. It was a revelation for some officials to see how their current zoning related to actual land use. In one town, these maps inspired a change in zoning to protect the area of a future water supply reservoir. These maps helped officials see how their towns fit into the regional picture and how their zoning and land use affected the adjoining towns, and vice-versa.
- *Performing "buildout" analysis:* A "buildout" analysis demonstrates the consequences of existing zoning. It assumes that all land that can be developed will be developed at some future date. In essence, it is a spreadsheet that divides the land available for development in each zone by the required lot size, subtracting

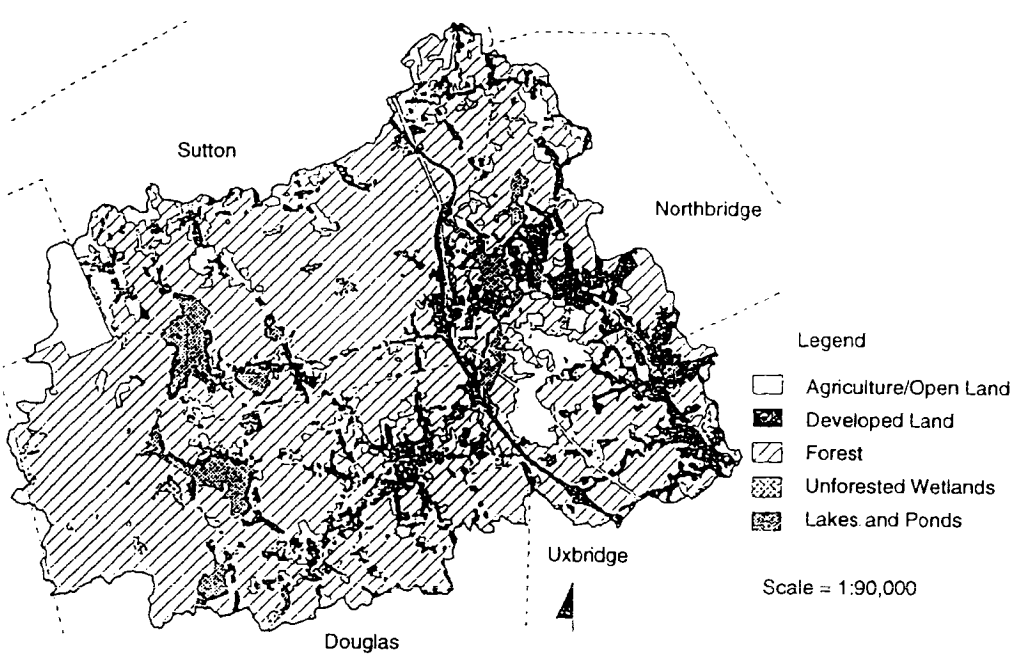

Figure 2. Land use/land cover map of Mumford River watershed.

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a certain percentage for the road network and steep slopes. It is best used to evaluate different development scenarios, substituting different zoning requirements.

- *Applying a watershed-wide pollutant loading model:* GIS provided the input needed to apply the "Simple Method" for estimating existing and potential pollutant loads. Future pollution loading was estimated using a buildout with existing zoning and again assuming the implementation of cluster zoning. The Simple Method was compared in one subwatershed with the Galveston Bay Method, which accounts for the hydrologic class of the soils.
- *Promoting planning for a greenway:* Land use maps were overlaid with parcel ownership to show the existing network of preserved open space and to identify those parcels of land having significant wildlife habitat and recreational value. In one town, these maps were used to gain funding for planning a river walk.

Database Development

The most daunting aspect of using a GIS is the prospect of spending a great deal of time and money creating a useful database. Fortunately for Massachusetts, many of the basic coverages needed for regional planning are housed in a state agency, MASS GIS, and are available for a small processing fee. These coverages include most of what appears on the standard United States Geological Survey (USGS) map: roads, streams, town boundaries, as well as watershed boundaries and land use data generated from the interpretation of aerial photographs. The university entered into an agreement whereby we gained access to this data at no charge, in return for sharing the new coverages that the project would generate.

New coverages needed for the study included: zoning, soils, sewer and water lines, and land ownership, or parcels taken from the assessor's maps. The soils maps were obtained from the SCS, digitized by hand, then the scale was converted with a computer program, "rubber-sheeting," to achieve a uniform scale of 1:25,000. All other new coverages were transferred onto a USGS topographical map at a scale of 1:25,000, then digitized directly into the computer. We obtained elevation data, but the triangulation process used to convert elevation data to slopes would require so much time and memory that, for our purpose, deriving a slope map from the four classes identified on the soils map was sufficient.

While GIS computer programs are powerful enough to perform most overlay and analysis functions necessary in nonpoint source pollution load modeling, database development and accuracy issues can limit the effectiveness of such modeling. The choice of which model to use is a function of which data are available for input. Physics-based distributed models are more precise but require detailed input parameters, beyond the scope of this project. The extent of our database limited us to lumped-parameter empirical models. We chose two such models, the Simple Method and the Galveston Bay Method.

GIS Applications

The Simple Method

Schueler (2) developed the Simple Method, one of the simplest lumped-parameter empirical models. The input data necessary to compute pollutant loading with the Simple Method are land use, land area, and mean annual rainfall. Land use determines which event mean concentration (EMC) values and percentage of imperviousness to use in the computation. The amount of rainfall runoff is assumed to be a function of the imperviousness of various land uses. More densely developed areas have more impervious surfaces, such as rooftops and paving, which cause stormwater to run off the land rather than be absorbed into the soil. The Simple Method can generate rough figures for annual pollutant loading within a watershed and can effectively show relative increases in pollutant levels as land is developed.

The formula used in the Simple Method is as follows:

$$L = [(P)(Pj)(Rv)/12] * (C) * (A) * (2.72)$$

$$(\text{load}) = (\text{runoff}) * (\text{EMC}) * (\text{area})$$

where:

L = pounds of pollutant load per year
P = rainfall depth (inches) over the desired time interval (1 year)
Pj = percentage of storms that are large enough to produce runoff (90 percent)
Rv = fraction of rainfall that is converted into runoff ($Rv = 0.05 + 0.009\ (I)$, where I represents the percentage of site imperviousness)
C = flow-weighted mean concentration (EMC) of the targeted pollutant in runoff (milligrams per liter)
A = area (in acres) of the study region

The Simple Method can be applied using a hand-held calculator or a computer spreadsheet program. For this project, the calculations were performed entirely within the ARC/INFO GIS environment, where the input data were stored. Results were exported to the Excel spreadsheet program for presentation purposes.

The application of the Simple Method consists of three major steps.

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Step 1: Aggregate Land Uses and Obtain Area Figures for Land Use Categories Within Each Subbasin

The land use coverage in our database has 21 categories. For the purpose of applying the Simple Method, these were aggregated into the following six major categories, based on development density: undeveloped forest and other open land, large-lot single-family residential, medium-density residential, high-density residential, commercial, and industrial. The aggregated land use categories were matched with study basins from the Nationwide Urban Runoff Program (NURP) for the purpose of assigning EMC values.

Step 2: Enter Percentage Imperviousness and Event Mean Concentrations for Each Land Use Type

The TABLES module of ARC/INFO was used to assign percentage of imperviousness and EMC values to individual land use polygons within the watershed's subbasins. The estimated percentage of imperviousness was obtained from Schueler's guide to using the Simple Method (2). EMC values for three pollutants—phosphorous, nitrogen, and lead—were taken from selected NURP study basins and were assigned to the aggregated land uses within the watershed.

Step 3: Input the Simple Method's Mathematical Loading Formula, Calculate Loading Results for Each Distinct Land Use Area, and Sum Results by Watershed Subbasin

Finally, the pollutant load was calculated for each distinct land use area within the Mumford River watershed by inputting the loading formula through the TABLES module of ARC/INFO. The mean annual rainfall figure was assumed to be that of Worcester, Massachusetts, or 47.6 inches. After calculating loading figures for phosphorous, nitrogen, and lead for each distinct land use area, these numbers were summed for each watershed subbasin, using the ARC/INFO frequency table reporting capability.

The Galveston Bay Method

As an experiment, we applied the Galveston Bay Method to one of the subbasins to compare results with the Simple Method. The slightly more sophisticated Galveston Bay model considers soil drainage characteristics in addition to land use/imperviousness to determine rainfall runoff. This method is similar to the Simple Method, in that amount of rainfall runoff and EMCs for particular land uses are multiplied by land area to determine total pollutant load (3). Runoff in this method, however, is calculated using the USDA SCS's TR 55 runoff curve model. The SCS model calculates runoff as a function of both land use and soil type. Runoff equals total rainfall minus interception by vegetation, depression storage, infiltration before runoff begins, and continued infiltration after runoff begins (4).

The formula used with the Galveston Bay Method has a structure similar to that of the Simple Method and is as follows:

$$L = \frac{P - 0.2\,[(1000/CN) - 10]^2}{P + 0.8\,[(1000/CN) - 10]} * EMC * A$$

(load) = (runoff) * (EMC) * (area)

where:

L = milligrams of pollutant load per year
P = mean annual rainfall amount
CN = runoff curve number, which is a function of soil type and land use
EMC = event mean concentration
A = area (in acres) of the study region

The application of the Galveston Bay Method consists of four major steps.

Step 1: Aggregate Land Uses and Obtain Area Figures for Land Uses Categories. Aggregate Soils According to Drainage Classes

Land use types were aggregated into the same six major categories as the Simple Method in order to match EMC values and to allow for later comparison of the two pollution loading methods. Soils were aggregated according to drainage classes for use with the USDA SCS TR 55 runoff formula. The SCS identifies four classes of soils according to their drainage capacity:

Class A = excessively to well-drained sands or gravelly sands.
Class B = well to moderately drained, moderately coarse soils.
Class C = moderately to poorly drained fine soils.
Class D = very poorly drained clays or soils with a high water table.

Step 2: Overlay Soils Data With Land Use Data and Clip This New Coverage Within the Subbasin

The ARC/INFO GIS overlay capability was used to overlay land use and soils maps for the Mumford River watershed on top of each other. This created new, distinct areas of different land use and soils combinations. Because we were only applying this model in one subbasin, the subbasin boundary was used in conjunction with the ARC/INFO "clip" command to cut out (like a cookie cutter) that portion of the watershed within the subbasin.

Step 3: Assign Runoff Curve Numbers and EMC Values to Each New Land Use/Soils Polygon Within the Subbasin

EMC values were assigned to each distinct land use/soils area in the same manner as they were assigned to land use areas using the Simple Method.

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Runoff curve numbers were assigned to each distinct land use/soils area within the subbasin according to values established by the USDA SCS.

Step 4: Calculate Loading Results for Each Distinct Land Use/Soils Area and Sum Results for the Entire Subbasin

Finally, the pollutant load was calculated for each distinct land use/soils area within the subbasin by inputting the loading formula through the TABLES module of ARC/INFO. After calculating loading figures for phosphorous, nitrogen, and lead for each distinct land use/soils area, these figures were then summed for the subbasin using the ARC/INFO frequency table reporting capability. Results of this modeling were converted from milligrams per acre per year to pounds per acre per year to facilitate later comparisons.

Buildout Scenarios

For planning purposes, GIS is most useful in its ability to quickly generate alternative scenarios. When these development scenarios are coupled with a pollutant load model as described above, alternative scenarios can be evaluated according to their impact on water quality. This project generated two different scenarios for each of the four towns in the watershed: a maximum buildout with existing zoning and a maximum buildout with clustered development.

Maximum Buildout

A maximum buildout scenario was used to show the worst case for development according to current zoning regulations (see Figure 3). The result of this buildout is expressed both in the number of new residential units to be built and in the area of land to be converted from undeveloped to residential and other urban uses.

Step 1: Eliminate From Consideration All Land That Is Already Developed

Step 2: Eliminate From Consideration All Land That Is Under Water

Step 3: Eliminate From Consideration All Land That Is Protected From Development

These protected lands included cemeteries, parks, and all land permanently restricted from development.

Step 4: Reduce the Remaining Amount of Land by 20 Percent To Account for New Roadways and Extremely Steep Slopes

The remaining land was considered to have "developable" status. Wetlands were included in this category because while a house probably would not be built on a wetland, wetlands can and often do constitute portions of the required lot size of large residential lots.

Step 5: Overlay the Land Use Coverage With Zoning and Minimum Lot Size Information

This created new land use areas as a function of zoning. All forests and fields were converted to a developed status.

Step 6: Divide Net Developable Land Area Within Each Zone by Minimum Lot Size Allowed To Obtain the Number of New Units

Results from the buildout are expressed in the number of new units. Results can also be shown spatially by shading in areas on the map according to future density

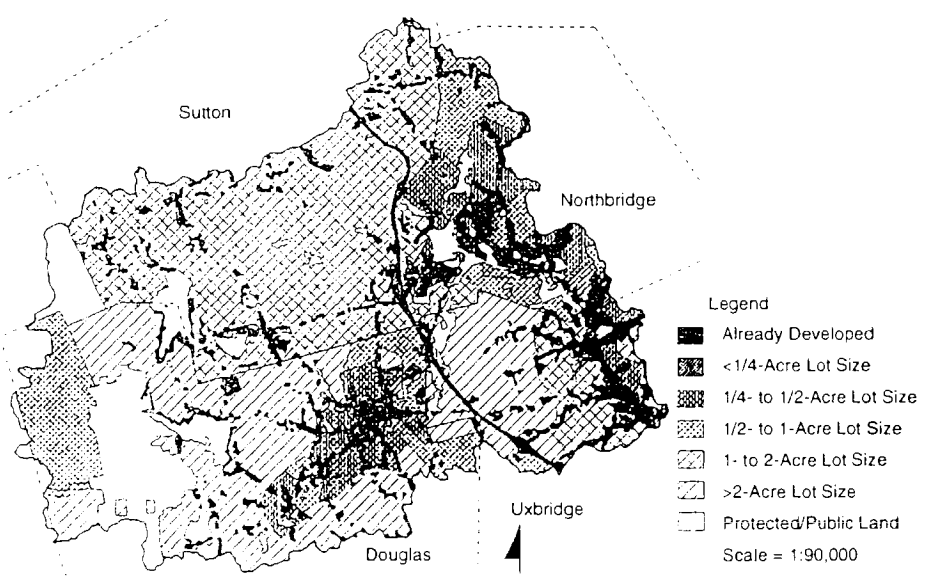

Figure 3. Maximum buildout scenario within the Mumford River watershed.

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of development (darker shades for higher density, lighter shades for lower density).

Clustered Buildout

Another alternative development scenario was generated assuming the implementation of clustered development. All areas zoned for lots larger than 1 acre were changed to cluster zones, where three-fourths of the land area remains undeveloped, and the remaining one-fourth of the land area is developed at a density of ½-acre lot size. With clustering, an area zoned for 2-acre house lots still supports the same number of new units, but three-quarters of the land area remains open space for passive recreation, protected wildlife habitat, and as a buffer zone to filter runoff.

Step 1: Select All Land Available for Development Zoned for 1-Acre Lots or Larger

Step 2: Multiply Selected Land by 0.75 and Add to the Category of Protected Land

Step 3: Multiply Selected Land by 0.25 and Change the Minimum Lot Size to One-Half an Acre

Step 4: Divide Step 3 by 20 Percent To Allow for New Roads and Steep Slopes

Step 5: Divide Step 4 by 21,780 (One-Half an Acre) To Determine Number of New Housing Units

Results

Lumped-parameter empirical models were chosen for this project and were applied to watershed subbasins ranging in size from 1 to 20 square miles and having an average of 4 square miles. The application of the Simple Method to existing land use conditions allowed for a comparison of the Mumford River watershed's subbasins for the purpose of identifying the subbasins that contribute the highest levels of pollutants per acre per year. The development of a maximum buildout scenario identified those areas within the watershed that will sustain the greatest amount of new growth. The application of the Simple Method to this maximum buildout scenario revealed that pollutant levels in surface water runoff would increase substantially for all subbasins in the watershed. This finding supports the theory of a positive relationship between development and increased pollutant levels from surface water runoff.

The development of a customized buildout scenario for future development identified those areas that are currently zoned for large-lot residential "sprawl" and that can support higher development density under cluster zoning, while protecting a significant amount of open space that can support a variety of beneficial uses. The application of the Simple Method to the customized buildout scenario revealed that the use of cluster development can reduce future levels of water pollution, especially from nutrients (see Figures 4 and 5).

Results determined by applying the Galveston Bay Method to one subbasin were compared with those obtained using the Simple Method. The predicted pollutant loading from current conditions differed significantly

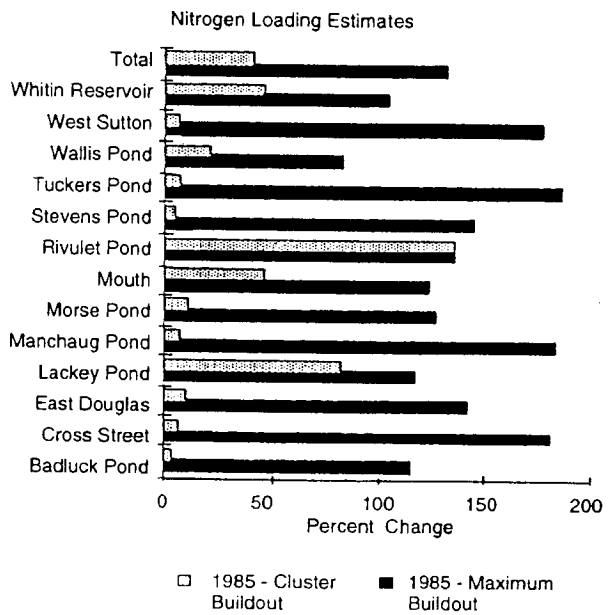

Figure 4. Chart showing difference in simple method results for nitrogen loading between maximum and customized buildout scenarios.

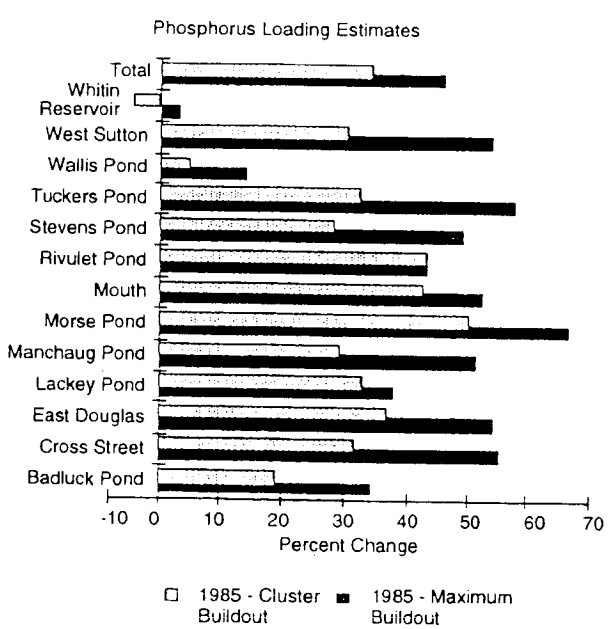

Figure 5. Chart showing difference in simple method results for phosphorus loading between maximum and customized buildout scenarios.

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between the two methods; the Simple Method consistently predicted five times the amounts generated by the Galveston Bay Method.

When the two methods were applied to both the maximum and customized buildout scenarios, however, the percentage growth of predicted pollutant loadings was remarkably similar for both methods; the Simple Method consistently predicted loadings 10 to 15 percent greater than the Galveston Bay Method. This indicates that while the Galveston Bay Method may provide more accurate results in predicting actual pollutant loading, the Simple Method is adequate enough for evaluating and comparing different development scenarios (see Figures 6 and 7).

Discussion

As states begin to implement a TMDL approach to regulating water quality, they face the quandary of how to determine the extent of nonpoint source pollution in the rivers. The crudest method is to subtract from the total load those quantities generated by point sources and call all the rest nonpoint source. While this is appropriate in some settings, it is unacceptable in a watershed with a long history of pollution because a significant source of pollution is the resuspension of historical sediments stirred up by storms. The situation demands the development of a model to predict the loading from nonpoint sources. Only a computer can handle the multiple factors that interact to generate nonpoint sources of pollution.

As greater emphasis is placed on watershed planning, the abilities of a GIS to input, store, manipulate, analyze, and display geographic information become indispensable. As the scientific community improves its knowledge base for determining the critical factors influencing nonpoint source pollution, GIS technology is improving in its ability to store and handle large amounts of data.

While a detailed, physics-based distributed model would be more accurate than the lumped-parameter models used for this project, they are difficult to apply at the watershed scale. The real limiting factor is the provision of all the data coverages needed to apply complex models. Lumped-parameter models, such as the Simple Method and the Galveston Bay Method, are ineffective for accurately predicting pollutant loads, but they are suitable for comparing and evaluating alternative development scenarios.

Time, and the development community, will not wait until all the answers are known. Local officials continue to approve development with no thought to the impacts on water quality. These officials need to be informed about the implications of haphazard growth. A GIS, with its ability to generate customized maps and quickly evaluate alternative development scenarios, is a powerful tool to help local officials visualize how the decisions they make on paper today will have an impact on the land tomorrow.

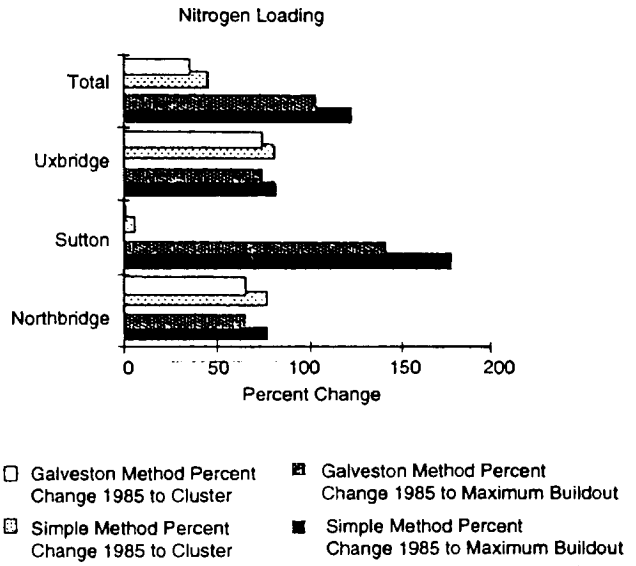

Figure 6. Chart showing difference between Simple Method results and Galveston Bay Method results for nitrogen loading.

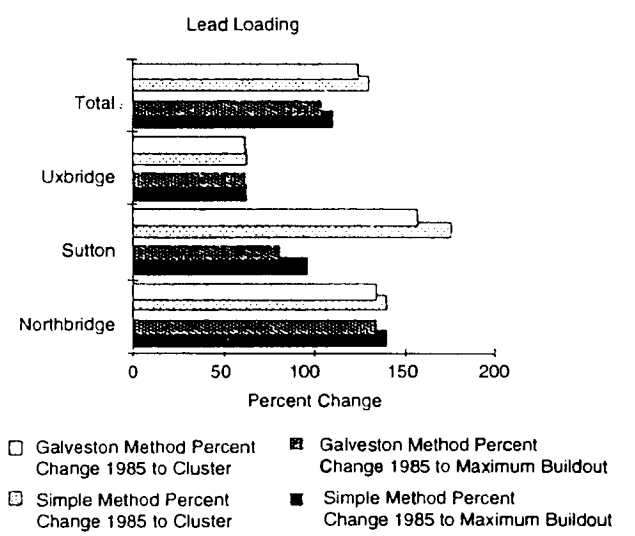

Figure 7. Chart showing difference between Simple Method results and Galveston Bay Method results for lead loading.

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A GIS for the Ohio River Basin

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Abstract

Much of the information used in the management of water quality in a river basin has a geographic or spatial component associated with it. As a result, spatially based computer models and database systems can be part of an effective water quality management and evaluation process. The Ohio River Valley Water Sanitation Commission (ORSANCO) is an interstate water pollution control agency serving the Ohio River and its eight member states. The U.S. Environmental Protection Agency (EPA) entered into a cooperative agreement with ORSANCO to develop and apply spatially based computer models and database systems in the Ohio River basin.

Three computer-based technologies have been applied and integrated: geographic information systems (GIS), water quality/hydraulic modeling, and database management.

GIS serves as a mechanism for storing, using, and displaying spatial data. The ARC/INFO GIS, EPA's agencywide standard, was used in the study, which assembled databases of land and stream information for the Ohio River basin. GIS represented streams in hydrologic catalog units along the Ohio River mainstem using EPA's new, detailed RF3-level Reach File System. The full Ohio River basin was represented using the less detailed RF1-level reach file. Modeling provides a way to examine the impacts of human-induced and natural events within the basin and to explore alternative strategies for mitigating these events.

Hydraulic information from the U.S. Army Corps of Engineers' FLOWSED model enabled EPA's WASP4 water quality model to be embedded in a menu-driven spill management system to facilitate modeling of the Ohio River mainstem under emergency spill conditions. A steady-state water quality modeling component was also developed under the ARC/INFO GIS to trace the movement and degradation of pollutants through any reaches in the RF1 representation of the full Ohio River basin.

Database management technology relates to the storage, analysis, and display of data. A detailed database of information on dischargers to the Ohio River mainstem was assembled under the PARADOX database management system using EPA's permit compliance system as the primary data source. Though these three technologies have been widely used in the field of water quality management, integration of these tools into a holistic mechanism provided the primary challenge of this study.

EPA's Risk Reduction Engineering Laboratory in Cincinnati, Ohio, developed this project summary to announce key findings of the research project, which is fully documented in a separate report of the same title.

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Introduction

During the past 25 years, computers have been actively used in water quality management, demonstrating their potential to assist in a wide range of analysis and display tasks. Technologies such as geographic information systems (GIS), database management systems (DBMS), and mathematical modeling have been applied in the water quality management field and have proven to be effective tools. For computers to achieve their full potential, however, they must become integrated into the normal programmatic efforts of agencies and organizations in the planning, regulation, and operational areas of water quality management.

Recognizing this need for routine use of computer-based tools, the Ohio River Valley Water Sanitation Commission (ORSANCO) and the Risk Reduction Engineering Laboratory (RREL) of the U.S. Environmental Protection Agency (EPA) commenced a study in 1990. The goals of the study included the adaptation, development, and application of modeling and spatial database management (DBM) tools that could assist ORSANCO in its prescribed water quality management objectives. These goals were consistent with EPA's ongoing programs involving the use of GIS and modeling technology. The study's goals also coincided with EPA's Drinking Water Research Division's work over the past decade, which applied similar technology to study the vulnerability of water supplies on the Ohio and Mississippi Rivers to upstream discharges.

Methodology Overview

To address the goals of this project, three basic technologies have been applied and integrated: GIS, water quality/hydraulic modeling, and DBM. GIS serves as a mechanism for storing, using, and displaying spatial data. Modeling provides a way to examine the impacts of human-induced and natural events within the basin and to explore alternative strategies for mitigating these events. DBM technology relates to the storage, analysis, and display of data. Though these three technologies have been widely used in the field of water quality management, integration of these tools into a holistic mechanism provided the primary challenge of this study.

GIS Technology

The guiding principle in developing the GIS capability was to maximize the use of existing GIS technology and spatial databases. The study used ARC/INFO GIS, EPA's agencywide standard. Remote access of ARC/INFO on a VAX minicomputer facilitated the initial work. Subsequently, both PC ARC/INFO and a workstation-based system were obtained.

EPA has developed an extensive spatial database related to water quality and demographic parameters. This served as the primary source of spatial data for the study. Following is a summary of spatial data used in this study:

- State and county boundaries.
- City locations and characteristics.
- Water supply locations and characteristics.
- Locations and characteristics of dischargers to water bodies.
- Toxic loadings to air, water, and land.
- Dam locations and characteristics.
- Stream reaches and characteristics.

The primary organizing concept for the water-related information was EPA's Reach File System (1). This system provides a common mechanism within EPA and other agencies for identifying surface water segments, relating water resources data, and traversing the nation's surface water in hydrologic order within a computer environment. A hierarchical hydrologic code uniquely identifies each reach. Information available on each reach includes topological identification of adjacent reaches, characteristic information such as length and stream name, and stream flow and velocity estimates. The original reach file (designated as RF1) was developed in the early 1980s and included approximately 70,000 reaches nationwide. The most recent version (RF3) includes over 3,000,000 reaches nationwide.

As part of this project, an RF1-level database was established for the entire Ohio River basin. The RF3 reach file was implemented for the Ohio River mainstem and lower portions of tributaries. River miles along the Ohio River were digitized and established as an ARC/INFO coverage to provide a linkage between the reach file and river mile indexing used by ORSANCO and other agencies along the river. Figure 1 shows the RF1 reach file representation of the Ohio River basin along with state boundaries.

The study incorporated several EPA sources of information on dischargers to water bodies. The industrial facility discharger (IFD) file contains locational and characteristic data for National Pollutant Discharge Elimination System (NPDES) permitted discharges. Detailed permit limits and monitoring information was accessed from the permit compliance system (PCS). The toxic release inventory (TRI) system includes annual loading of selected chemicals to water, land, air, and sewer for selected industries based on quantity discharged. All water data are referenced to the NPDES permit number, which is spatially located by reach and river mile, and by latitude and longitude.

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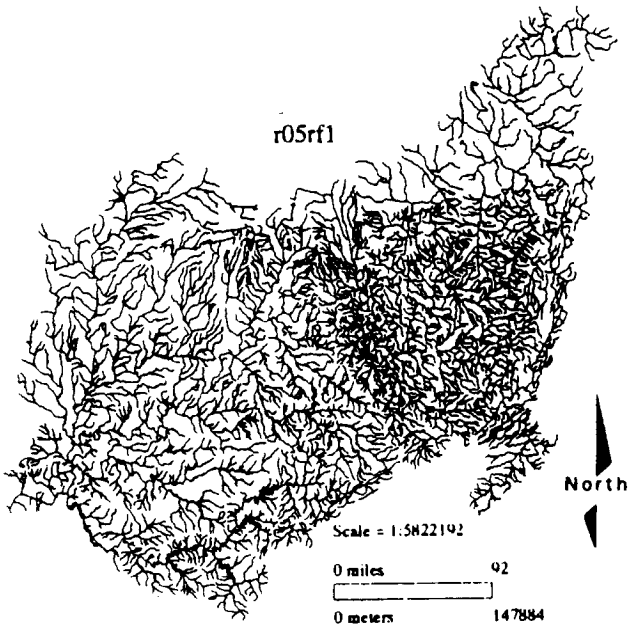

Figure 1. RF1 reaches in the Ohio River basin.

Spill Modeling

An important role that ORSANCO fills on the Ohio River relates to the monitoring and prediction of the fate of pollutant spills. Typically, ORSANCO serves as the overall communications link between states during such emergency conditions. ORSANCO coordinates and participates in monitoring and serves as the information center in gathering data and issuing predictions about the movement of spills in the river. In the past, a series of time-of-travel nomographs, based on National Weather Service flow forecasts, Corps of Engineers flow-velocity relationships, and previous experience, were used to predict the movement of spills. This project combined a hydraulic model with a water quality model to serve as a more robust method for making such predictions.

The U.S. Army Corps of Engineers' FLOWSED model was selected as the means of predicting daily flow quantities and water levels along the mainstem and portions of major tributaries near their confluence with the Ohio River (2). The Ohio River Division of the Corps of Engineers applies FLOWSED daily as part of its reservoir operations program. The Corps can generate 5-day forecasts of stage and flow for 400 mainstem and tributary segments, and ORSANCO can access the results via phone lines.

EPA's WASP4 water quality model was selected for use in the project (3). WASP4 is a dynamic compartment model that can be used to analyze a variety of water quality problems in a diverse set of water bodies. Because the primary use of the model in this project is quick response under emergency situations, only the toxic chemical portion of the model with first order decay is being used. The FLOWSED and WASP4 models have been combined into a user-friendly spatial decision support system framework described later in this project summary.

Discharger Database Management System

EPA's PCS and historical records maintained by ORSANCO furnish a rich source of data on discharge information for the Ohio River. To organize these data and make them available for analysis, a database was developed using the PARADOX DBM system.

The database was established using a relational structure with a series of related tables (two-dimensional flat files). Individual tables contain information on facilities, outfalls, permit limits, monitoring data, and codes used in the other tables. The NPDES permit number is used as the primary key in each data table. A mechanism for downloading and reformatting data from the national PCS database has been developed along with custom forms for viewing and editing data, and custom reports for preparing hard copy summaries. Latitude and longitude values for each facility can provide the locational mechanism for use of this data in conjunction with GIS.

Integration of GIS/Modeling/Database Technologies

A major objective of this study was the integration of GIS, modeling, and DBMS technologies into a holistic tool for use by ORSANCO. Several integration mechanisms were implemented as summarized below.

Steady-State Spill Tracing

The NETWORK component of the ARC/INFO GIS provides a steady-state, transportation-oriented routing capability. This capability was used in an arc macro language (AML) program to construct a routing procedure for determining downstream concentrations and travel times. The pollutant may be treated as a conservative element or represented by a first order exponential decay function. This capability has been implemented for use with the RF1 reach file representation of the full Ohio River basin. The user may select from six flow regimens: average flow, low flow, and four multiples of average flow ranging from one-tenth to 10 times average flow. This system gives ORSANCO the ability to estimate the arrival time of a spill from any RF1 tributary to the Ohio River mainstem.

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Spill Management System

A PC-based spatial decision support system (SDSS) was built as a spill management system to be a quick response tool for analyzing and displaying the results of pollutant spills into the Ohio River. The schematic in Figure 2 illustrates the components in this computerized spill management system. The system is implemented in the C language using a commercial menuing system and a series of graphic display routines developed at EPA. Custom, written routines have been used to read the output from the U.S. Army Corps of Engineers' FLOWSED model, to generate input files for EPA's WASP4 model, to create output reports and output plots, and to provide an animated representation of the concentration profiles moving down the river. Figure 3 presents an example of a graphic output the system generated. Additionally, the system generates a file in

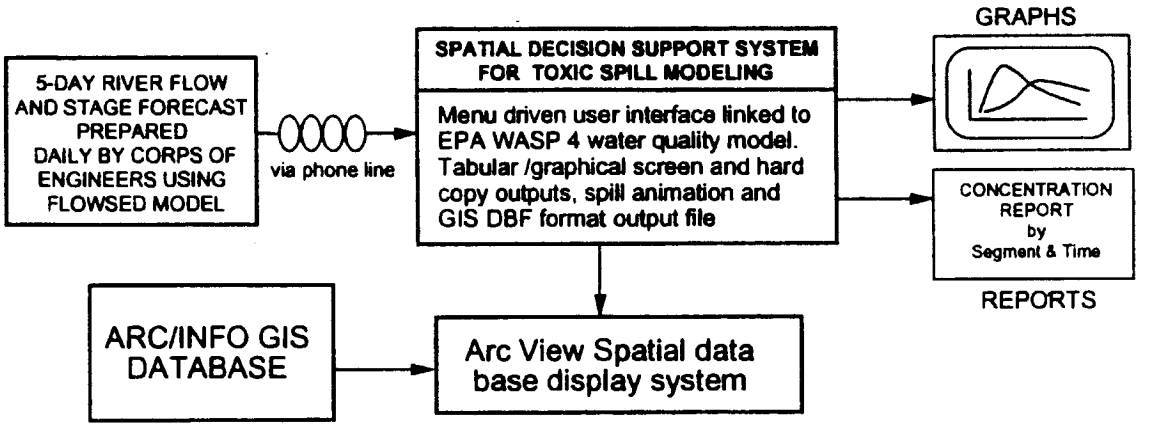

Figure 2. Schematic representation of spill modeling system process.

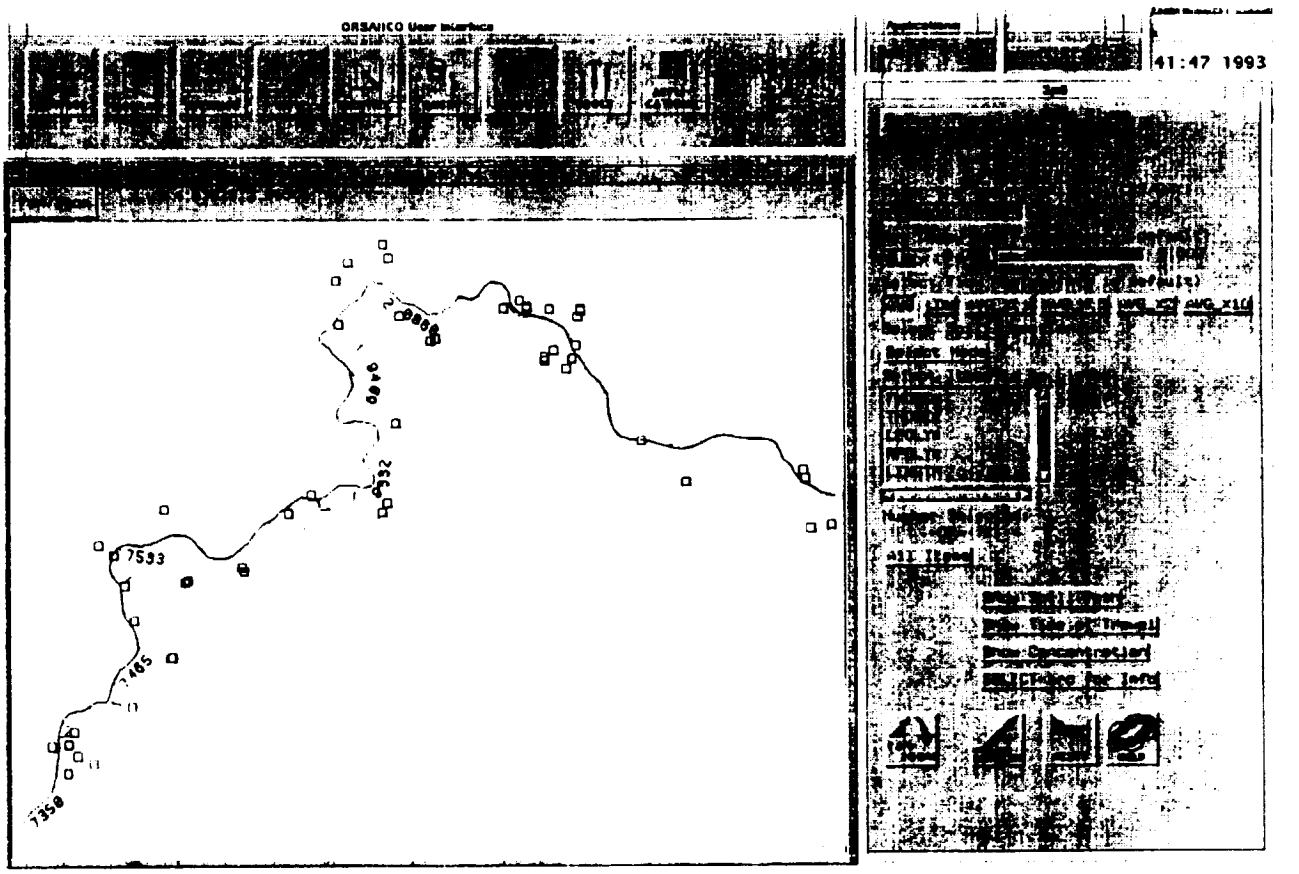

Figure 3. Graphic output from the basinwide network spill model.

DBF format that may be read by ARC/VIEW (the companion software to ARC/INFO for user-friendly viewing of spatial data).

Hardware Platform

Within the study, the initial hardware platform was a combination of local PCs (in Cincinnati) and a remote access terminal to a VAX computer located at EPA's National Computer Center in Research Triangle Park, North Carolina. The final platform, and the one on which the completed system was installed, comprised a UNIX-based Data General workstation and a PC workstation. The full hardware configuration is shown schematically in Figure 4.

Conclusions

The application of computer-based display, analysis, and modeling tools in conjunction with GIS technology proved to be an effective strategy for water quality management. This study used an existing GIS package and DBMS in conjunction with existing water quality and hydraulic models. The study focused primarily on assembling available spatial and relational databases and integrating the systems to provide a usable, effective tool.

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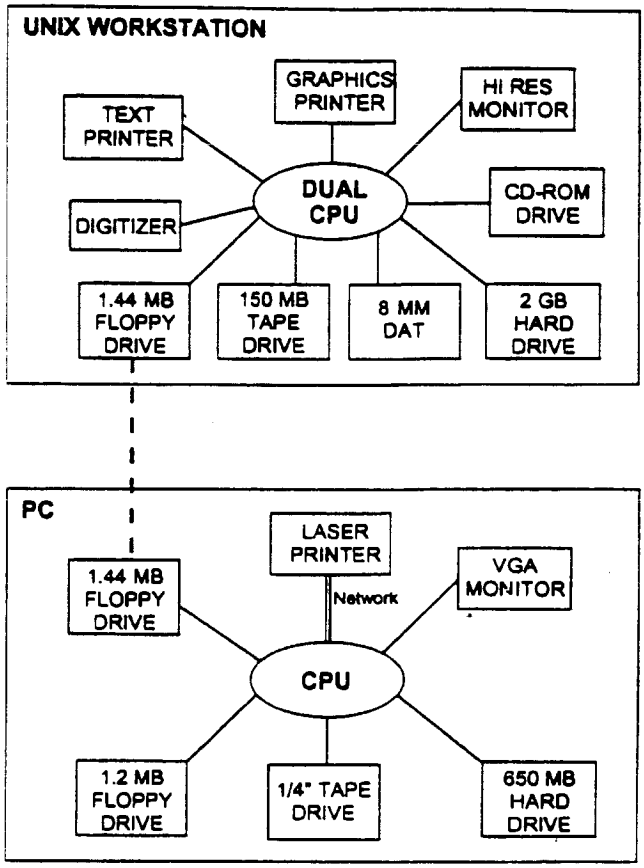

Figure 4. Hardware configuration.

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Nonpoint Source Pesticide Pollution of the Pequa Creek Watershed, Lancaster County, Pennsylvania: An Approach Linking Probabilistic Transport Modeling and GIS

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Abstract

The U.S. Environmental Protection Agency (EPA) has mandated that each state prepare a state management plan (SMP) to manage pesticide residues in the state's environment. One aspect of an SMP involves identifying specific soils and sites that may be vulnerable to the transport of pesticides into water resources. A recently developed system identifies vulnerable areas by coupling probabilistic modeling that uses the Pesticide Root Zone Model (PRZM) with a desktop geographic information system (GIS-MAPINFO). A limited test of this system succeeded in identifying and mapping individual soil series in a watershed that were shown to have transported atrazine to surface and ground water.

During this project, various digital data sources were evaluated for availability and ease of use, including:

- STATSGO.
- U.S. Geological Survey (USGS) digital line graphs (DLGs).
- National Oceanic and Atmospheric Administration (NOAA) climate data.

This study documents hands-on hints and tricks for importing and using these data.

From 1977 to 1979, the USGS measured the movement of atrazine off fields of application into water resources in the Pequa Creek basin in Lancaster County, Pennsylvania (1). Atrazine in surface water appeared at levels exceeding 20 parts per billion in storm flow and above the 3 parts per billion maximum contaminant level (MCL) during base flow from Big Beaver Creek, a tributary to Pequa Creek. Each soil series in the subbasin was digitized into a GIS. PRZM allowed simulation of runoff, erosion, and leaching of atrazine (applied at 2.24 kilograms per hectare in conventionally tilled corn) for each soil. This process included simulating each soil under different slopes for an 11-year period from 1970 to 1980. Interpreting the results for each soil series determined the probability distribution of atrazine in kilograms per hectare for each mode of transport. GIS used these data to thematically map each soil series for atrazine loss.

The results of this demonstration project suggest that the Manor silt loam, with slopes varying from 6 percent to 20 percent, had a high potential to transport atrazine residues to surface water. This type of analysis could suggest that this soil series be:

- Farmed using conservation tillage.
- Managed to install grass waterways or buffer strips to stop runoff.
- Set aside from production to protect water resources.

Digital databases were available for the study area, but many technical problems were encountered in using the data. Researchers embarking on these types of modeling and GIS projects should prepare themselves for significant expenditures of time and finances.

Introduction

A significant volume of published literature documents pesticide residues in ground water, and the volume of investigations of residues in surface water is expanding. The growing acceptance of immunoassay techniques for the determination of pesticide residues in water has given the field of pesticide monitoring an accurate and economical analytic methodology. This will result in an increase in monitoring capability at the federal, state,

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local, and university levels. These increases in monitoring capability have documented and will continue to document the occurrence of pesticides in water resources as the result of past transport through the soil profile. The U.S. Environmental Protection Agency (EPA) has mandated that each state prepare a state management plan (SMP) to manage pesticide residues in the state's environment. Lacking, however, is a reliable pesticide screening technique to indicate which soils, on a countywide scale, may be sensitive to the transport of a specific pesticide to deep within the soil profile or to the surface water resources. These assessments would greatly supplement the usability and validity of SMPs.

Electronic databases such as State Soils Geographic (STATSGO), Data Base Analyzer and Parameter Estimator (DBAPE), or the SOIL5 subsets found in Nitrate Leaching and Economic Analysis Package (NLEAP) provide easy access to detailed soil data and model input estimator subroutines, thereby simplifying data entry to numerical models. Two groupings define soils: soil series and soil associations. Soil series are the individual soil taxa found in a field. Soil associations represent groups of soil series, usually three or four soil series occurring together in an area, and are mapped as a single unit on a county scale. Mapping of most soil associations across the United States is complete, with open access to the county scale maps. A digital soils mapping data set called SSURGO contains many of the soil series maps for the United States. Climatologic databases also provide easy access to long-term data from the National Oceanic and Atmospheric Administration (NOAA) weather stations, allowing a user the opportunity to input realistic climate data to pesticide transport models.

Many numerical pesticide transport models, such as the Pesticide Root Zone Model (PRZM), Ground-Water Leaching Effects of Agricultural Management Systems (GLEAMS), and Leaching Estimation and Chemistry Model (LEACHM), can produce transport estimates for specific pesticides in specific soils. Each model has its own strengths and weaknesses, and detailing these characteristics is beyond the scope of this paper. Several authors, however, have described comparisons between models (e.g., Smith et al. [2], Mueller et al. [3], and Pennell et al. [4]). These numerical models all generally require extensive site-specific soil, agronomic, and climatologic databases. The results from these models are extremely detailed. Their pesticide transport estimates, however, are only valid for those locations for which site-specific data are sufficient to allow calibration of the model. Applying such site-calibrated model results to larger scales (county scales) is inappropriate.

In one procedure, a user could identify soils and use transport models that may have a limited ability to retard pesticides from reaching water resources. This type of modeling has recently been called probabilistic modeling (5). The concept behind this procedure is to use an existing transport model, such as PRZM, and vary certain input parameters (e.g., slope, organic content, pesticide K_{oc}) to produce a probability of a given output being equaled or exceeded. For example, a PRZM model could be created for a soil series with an average organic content of 1 percent and a slope of 8 percent in the eastern corn belt. The model would use the 30 years of historical climate data for a nearby station. The model would vary the organic content and surface slope within given ranges for the soil series and run 1,000 simulations. The analysis could then entail plotting the results (i.e., monthly runoff loads, erosion loads, and leaching through the root zone) in a frequency diagram and generating probability curves. This analysis would allow the user to estimate the anticipated pesticide losses, runoff, erosion, and leaching for any given soil in the county. The soils with greater probabilities for pesticide loss could be identified and mapped using GIS.

Recent advances in computing speed and efficiency have reduced the amount of time and expense needed to run numerical pesticide transport models. This makes it possible, in a relatively short amount of time, to quantitatively model not just one soil series, but the hundreds of major soil series that occur in an entire state (e.g., 357 major soil series combined into 464 different soil associations in Wisconsin). This type of model can be very useful to the development of SMPs as well as to a variety of users, including pesticide registrants, bulk pesticide handlers, custom appliers, county agricultural extension agents, and individual growers.

Objective

The objective of this study was to use probabilistic modeling analyses and a geographic information system (GIS) to determine which soil series in a watershed may contribute to nonpoint source pollution through runoff of agricultural chemicals. Specifically, the study aimed to locate a watershed with historical atrazine runoff, map the soils, and perform transport modeling using historical precipitation. The results of this procedure would determine which soil series had a high potential to contribute to the nonpoint source pollution of the watershed. Once the transport modeling was completed, a GIS would help map the distribution of the sensitive soil series. The mapping would act as a base for implementing best management practices (BMPs) to reduce nonpoint source pollution.

Background

Ward (1) described the water quality in the Pequa Creek basin in Lancaster County, Pennsylvania, for the years 1977 through 1979. Flow from Pequa Creek (154-square-

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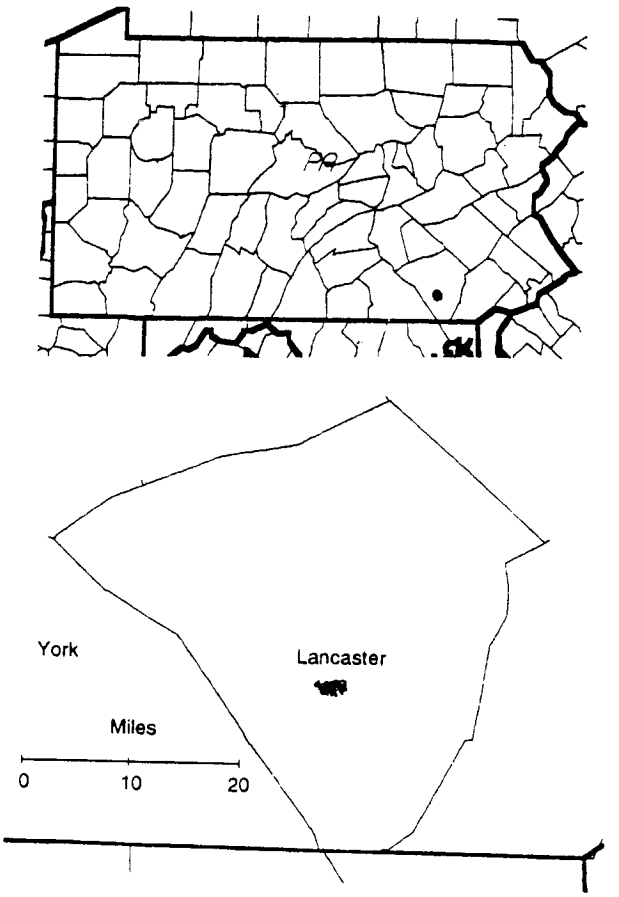

Figure 1. Location of Pequa Creek basin, Lancaster County, Pennsylvania.

mile drainage area) eventually discharges into the Chesapeake Bay (see Figure 1). The data collection efforts (6) documented the occurrence of atrazine (2-chloro-4-ethylamino-6-isopropylamino-s-triazine), a commonly used herbicide for weed control in corn-growing regions, and other agrichemicals in both base-flow and storm-flow conditions of the Pequa Creek. A subbasin of Pequa Creek, Big Beaver Creek, had the greatest reported atrazine concentrations during the sampling period. The maximum reported atrazine concentrations at the Big Beaver Creek sampling station, near Refton, Pennsylvania, were 0.30 parts per billion during base-flow conditions and 24.0 parts per billion during storm-flow conditions. The Big Beaver Creek basin is 20.4 square miles in area, and agriculture constituted about 66 percent of the land use in 1979. Corn was grown on 26.6 percent of the agricultural lands in this subbasin. The average rainfall for this basin is about 37 inches annually (1).

As noted, agriculture represented the major land use in the area. The primary agricultural soils in Lancaster County are silt loams (Typic Hapludults and Hapludalfs) in texture with slopes that range from 0 percent to 8 percent (7). Upon inspection of the air-photo soil series maps found in the county soil survey, however, agricultural crops grew on lands with slopes of up to and exceeding 15 percent, with soils such as the Manor silt loam, Pequa silt loam, and Chester silt loam (7).

Natural soil organic contents in the agricultural soils range from 0.1 percent to 2.0 percent. Water contents of the agricultural soils range from 10 percent to nearly 30 percent. The soil Erosion Factor (K) for the surface layer ranges from 0.17 for the relatively stable Ungers Series to 0.43 for the Pequa Series. The greater the value, the greater the susceptibility to sheet and rill erosion. The soil Erosion Factor (T in tons/acre/year) for the entire soil profile ranges from 2 for the relatively stable Clarksburg Series to 5 for the Elk Series.

Methods

Determining which soil series in the Big Beaver watershed may contribute to nonpoint source pollution through runoff of agricultural chemicals entailed performing a combination of probabilistic modeling analyses and a GIS data manipulation.

Physiographic and Soil Series Boundaries

The orientation of Pequa Creek, Big Beaver Creek, and other surface water bodies was digitized directly from the 1:50,000-scale county topographic map for Lancaster County (8). This map also provided the basis for digitizing the Pequa Creek drainage divide, location of urban areas, and roadways. The MAPINFO GIS allows for the creation of boundary files by tracing the boundary off the topographic map with a digitizing tablet configured to the latitude and longitude coordinates of three points on the map. The latitude and longitude are displayed while the boundary is being traced, allowing the user to verify the accuracy of the boundary against known coordinates on the map. GIS contains a self-checking boundary closure program to ensure that the polygons are closed and that the boundary contains no extraneous line segments. These boundary data are already available from the USGS in a digitized format, digital line graphs (DLGs). Because digitizing is an easy task, however, and to minimize costs, the project used manual digitizing rather than purchase the data.

The roadways and urban areas were digitized to allow use of standard control points, such as road intersections and benchmarks, to configure the U.S. Department of Agriculture (USDA) Soil Conservation Service air-photo-based, 1:15,840-scale soil series maps for the Big Beaver Creek watershed. Known land grid coordinates were noted on the air-photo maps (7). We concluded, however, that using known reference points, such as roadways and towns, allowed for a better configuration of the digitizing tablet to the air photos and

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eliminated concerns over scale distortions sometimes common in air photos.

After configuring the air photos to the digitizing tablet, a 2-square-mile area around the surface water sampling points was digitized. The next step entailed digitizing all the mapped soil series units within this area. The locations of crop areas, as plowed fields, were noted. In noting forested areas, it became apparent that only minor acreages were not in agricultural production. Those mapped units were generally the Manor and Pequa Series soils with slopes exceeding 25 percent.

Pesticide Transport Modeling

The PRZM pesticide transport model helped to quantify the ability of several soil series to retard the transport of atrazine through the root zone as leachate, dissolved in surface runoff and adsorbed on sediment that moved during erosion. The PRZM model performed in an uncalibrated or screening model mode. The input values for soil properties came from both the EPA DBAPE database and the Lancaster County Soil Survey (7). The modeled soil profile was 150 centimeters thick and divided into 5-centimeter compartments. The soil half-life of atrazine was set at 57 days in accordance with values that the PRZM manual listed (9). The primary soil property that varied in this demonstration project was surface slope. All other parameters, such as soil organic content, moisture content, and bulk density, appeared as midpoint values for the ranges listed in DBAPE.

The agronomic scenario that the model simulated was for corn grown continuously for 10 years using conventional tillage practices and planted on May 7 of each year. Atrazine was surface applied at a rate of 2 pounds per acre (2.24 kilograms per hectare) on May 1 of each year. For climatic input, the model used the historical precipitation regimen from 1970 through 1980, as measured at the Harrisburg, Pennsylvania, station.

PRZM simulations were made for each of the following soil series:

- Chester
- Conestoga
- Elk
- Glenelg
- Glenville
- Hollinger
- Letort
- Manor
- Pequa

Monthly values were calculated for leachate, runoff, and erosion per hectare. Unfortunately, no data for the Pequa Series were available in the DBAPE database; therefore, this portion of the analyses omitted it. In addition, analyses of the Manor Soil Series included more detailed probabilistic modeling where the surface slope held constant (6-percent slope) and the surface soil organic content varied to include the high, average, and low organic contents as listed in DBAPE. PRZM also calculated the volume of water as evapotranspiration, runoff, and recharge through the root zone.

Results

The results of this study should demonstrate the application of transport modeling to the possible protection of water resources. Regulatory decision-makers should not consider these results in their current form because such decisions would require a much more rigorous simulation strategy to increase the level of confidence in the data. As a demonstration study, however, the results do show the usefulness of this approach. Table 1 contains the cumulative frequency data for the simulated atrazine residues in runoff, erosion, and leaching that occurred under 30 years of historical precipitation. The data cover the 12 soils mentioned, with the surface slope held constant at 6 percent.

Atrazine in Runoff and Erosion

The results of this analysis suggest that the Haggerstown Series had the greatest potential for yielding atrazine in runoff; approximately 50 percent of the simulated monthly atrazine in runoff values equaled or exceeded 0.0001 kilogram per hectare. Conversely, the Elk Series yielded the least atrazine to runoff; 50 percent of the runoff data were at residue levels of 1×10^{-6} kilograms per hectare. Within the Big Beaver Creek subbasin, the Manor Series had the greatest potential to yield atrazine in runoff.

As with the runoff data, the Haggerstown Series had the greatest potential to yield atrazine in eroded sediments, and the Elk Series yielded the least atrazine in erosion. Within the Big Beaver Creek subbasin, the Manor Series had the greatest erosion potential regarding atrazine.

GIS Analyses

After entering the results from the transport modeling into a database, GIS could produce maps showing the location of soils with high runoff potentials. Figure 2a shows the orientation of soil series around the surface water sampling points in Big Beaver Creek. Figure 2b represents the same scene but fills in the soils with high runoff potential. Using this type of analysis can help areas that may be sources of nonpoint source runoff contamination. Once identified, these soils can be targeted for alternative management practices that may reduce the amount of runoff and the degree of nonpoint source contamination.

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Table 1. Cumulative Frequency of Simulated Atrazine Residues in Runoff for 12 Major Soils in Lancaster County, Pennsylvania (values are percentage of data)

Load (kilograms per hectare)	Soil Series in Lancaster County, Pennsylvania											
	Bucks	Chester	Clymer	Connestoga	Elk	Glenelg	Haggerstown	Hollinger	Lansdale	Letort	Manor	Ungers
IE-10	83.33	84.85	84.09	84.08	78.79	84.85	93.94	83.33	88.64	85.61	86.36	84.09
IE-9	78.79	80.30	80.30	81.82	75.76	80.30	93.94	81.06	85.61	84.85	84.61	84.09
IE-8	74.24	76.52	75.00	78.03	69.70	76.52	93.94	77.27	80.30	84.09	84.85	81.82
IE-7	66.67	68.94	66.67	72.73	59.09	68.18	91.67	68.94	75.00	83.33	75.52	75.76
IE-6	56.06	46.97	56.06	61.36	50.00	56.06	84.85	59.09	65.15	76.52	66.67	67.42
IE-5	47.78	35.61	46.21	50.00	43.18	47.73	74.24	47.73	51.52	67.42	46.97	53.03
IE-4	36.36	21.97	32.58	35.61	31.82	34.85	53.79	34.85	40.15	46.97	31.82	39.39
IE-3	24.24	15.15	21.21	21.21	21.21	21.21	37.12	21.21	24.24	31.82	16.67	24.24
0.00	15.91	6.82	14.39	15.12	15.15	13.64	19.70	15.15	15.91	17.24	4.55	15.15
0.01	6.82	0.00	6.06	6.06	6.82	3.79	7.58	6.06	6.82	4.55	0.00	4.55
0.10	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
10.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
25.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

This table reads as follows: Given the Elk Series, the first value reads that 78.79 percent of the simulated data were greater than or equal to IE-10 kilograms per hectare. Similarly, within the same column, 6.82 percent of the data were greater than 0.01 kilograms per hectare but less than 0.1 kilograms per hectare.

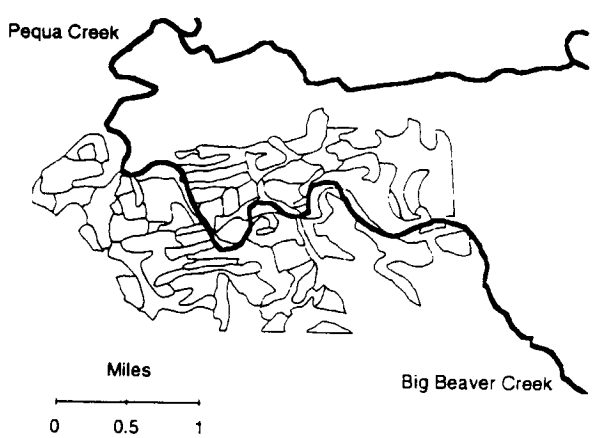

Figure 2a. Soil series in the Big Beaver Creek basin.

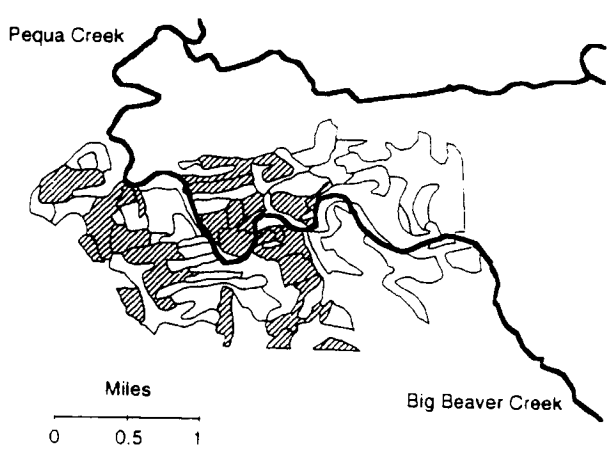

Figure 2b. Soils sensitive to atrazine runoff in the Big Beaver Creek basin.

Detailed Modeling of the Manor Series

Performing an introductory probabilistic modeling exercise allowed further investigation of the potential of the Manor Series to release atrazine into runoff. The existing 30-year climate data and the stated agronomic data were retained from the previous modeling. The organic carbon content of the surface soil layer, however, was allowed to vary between the published low, average, and maximum values found in the DBAPE database. This exercise followed the principles set forth by Laskowski et al. (5) and others who describe probabilistic modeling approaches. In essence, by varying input parameters within known endpoints, the probabilistic approach can generate a distribution of pesticide residue values that statistically reflects the anticipated residues. Parameters to vary may include:

- Organic carbon content
- Surface slope
- Kd (distribution coefficient)

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- Moisture content

By allowing input variables to vary according to a normal distribution, this approach thereby eliminates some of the uncertainty associated with pesticide transport modeling. The probabilistic modeling approach requires the creation of a significant database by performing many runs (e.g., 1,000 model runs that generate 12,000 monthly values for each soil).

This study included a limited probabilistic modeling exercise. Table 2 lists the results for the mean atrazine residues in runoff, erosion, and leaching for the Manor Soil Series during the:

- Entire year
- Growing season
- Winter months

The surface slope was held constant at 6 percent, but the soil organic carbon content varied within the published range. The means for all months show limited variation in mean residues. Runoff was by far the major source of atrazine. Erosion and leaching values were on similar scales (trace amounts).

Table 2. Summary Statistics for Detailed Modeling of the Manor Soil Series in the Pequa Creek Watershed (statistics based on 1,080 values)

Percent Organic Carbon[a]	Mean Atrazine Residue (kilograms per hectare)		
	Runoff	Erosion	Leaching
All Months			
Low	0.01381	0.00024	0.00028
Average	0.01229	0.00042	0.00012
High	0.01105	0.00057	< 0.000006
Growing Season			
Low	0.03290	0.00058	0.00017
Average	0.02881	0.00100	< 0.000008
High	0.02540	0.00130	< 0.000004
Winter Months			
Low	0.00014	< 0.000002	0.00036
Average	0.00045	< 0.000001	0.00014
High	0.00075	< 0.000003	< 0.000007

[a]Data taken from DBAPE soils database as low, midpoint, and maximum reported organic contents.

The greatest runoff and erosion values occurred during the growing season. The greatest leaching, however, occurred during the winter months. These results support the general observations that surface residues run off during spring and summer but that as the crops grow and evapotranspiration increases, recharge to ground water decreases, subsequently limiting pesticide transport to ground water. Conversely, during the winter months, the surface soil pesticide residues generally decrease because of exposure to months of photolysis, hydrolysis, and biodegradation. Subsurface residues have been protected from degradation, however, and increased ground-water recharge, due to great reductions in evapotranspiration, transports the residues through the soil column.

This limited exercise provided a valuable learning experience regarding probabilistic modeling. As computing techniques and hardware advance, the cost in time and money for each simulation should decrease dramatically. Although researchers tend not to have great faith in pesticide transport modeling, the advances in this field will reduce uncertainty and instill greater confidence in the modeling process.

GIS Pitfalls

GIS is a powerful tool and has great promise for use in environmental problem-solving. Several points or pitfalls, however, hinder broad acceptance of GIS. As with most new technologies, cost is the overriding concern in using GIS. Although technical staff and project scientists understand the power of GIS and the effort that data preparation requires, management and corporate staff often do not see the benefits for the costs. Many managers assume that current GIS systems resemble those seen on "Star Trek," and when reality becomes apparent, managers tend to discard GIS as too costly and complex. Several points need consideration when contemplating the use of GIS. Although various products exist, this discussion focuses on ARC/INFO and MAPINFO products.

Hardware

Computer hardware is plentiful if the available budget can support a purchase. Many high- powered GIS packages (e.g., GRASS, ARC/INFO, INTERGRAPH, IDRISI) run best on mainframes or minicomputers. Most technical staff, however, only have access to PC machines. Corporate purchasing departments more readily expend funds for PC technology because they will eventually find use for these machines even if they are not used for GIS. A recent ARC/INFO advertisement (August 1994) lists costs for SUN SPARC minicomputer systems with ARC/INFO software at $12,000 to $15,000 depending on configuration.

Minicomputers and mainframes require specialized staff to configure and maintain the hardware. Today, many staff level personnel can open and augment their PC machines with a minimum of external support. GIS performance reflects the tradeoff in hardware, particularly

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when a considerable amount of data manipulation is required. For example, if linking discrete depth soil series data to STATSGO soil associations is necessary, then a minicomputer system may be best. The postprocessed data could, however, be exported to a format that will run on PC-based systems. If the user wants to import and manipulate remote sensing imaging (e.g., SPOT or Landsat data), then minicomputers are recommended. If the user wants to display already edited images and preprocessed GIS data, then PC-based computing may be sufficient. The ultimate use of GIS drives the hardware selection.

Software

A great number of GIS software packages are available to meet almost any level of use and expertise. Software runs under both UNIX and DOS/Windows (denoted as DOS for the remainder of this paper) operating systems. The UNIX-based software tends to be more powerful and flexible than the DOS-based software. UNIX-based packages require more specialized staff to optimize GIS, however.

UNIX-based software packages include GRASS, ARC/INFO, INTERGRAPH, and IDRISI. Costs vary from public domain charges for GRASS and IDRISI to vendor supplied ARC/INFO and INTERGRAPH, which can cost several thousand dollars each.

DOS versions of ARC/INFO (e.g., PC ARC/INFO, ARCAD, ARC/VIEW) are also available and provide the user with various levels of data editing and manipulation abilities. Generally, PC ARC/INFO is the same as the UNIX version, varying in speed of processing. ARCAD is a GIS engine that uses AutoCAD for drawing and displaying, giving the user most of the abilities of the UNIX-based version. ARC/VIEW I was primarily a display and simple analysis tool. It allowed the user to view, display, and manipulate existing arc data but did not support image editing. Currently, ARC/VIEW II provides more support for image editing and data manipulation. Costs range from about $3,000 for PC ARC/INFO and ARCAD (AutoCAD also costs about $2,000) to around $500 for the ARC/VIEW products.

MAPINFO is a DOS-based GIS that was designed for marketing and demographic applications. Several researchers, however, have used MAPINFO for environmental applications. The most outstanding feature of MAPINFO is that it easily imports data layers as it reads dBASE type files directly. MAPINFO V3 also reads database files and recreates them as *.TAB files. In contrast to the "coverage and entity" concepts of the ARC/INFO line of programs, MAPINFO reads latitude and longitude coordinates and displays the results. This simplifies data management because many researchers who have already created custom databases can easily import those data as long as latitude and longitudes coordinates are present.

As with the ARC/INFO line of programs, many common data layers can be purchased for use in MAPINFO. These layers can be expensively priced, costing approximately $1,000 per county for roadway, census, and demographic data. One major lapse is the poor library of environmental layers, USGS topography, hydrography, soil boundaries, or climate stations. MAPINFO does sell a module that allows users to convert to and from ARC/INFO coverages so that common data layers can be established. Experience shows, however, that conversion programs do not always work as advertised. For example, large boundary files (STATSGO data for Indiana) do not readily convert from ARC/INFO to MAPINFO. Third-party vendors may be needed to convert data for use in MAPINFO.

One very important factor supporting the use of MAPINFO is that it has a business application slant; therefore, it is slightly easier to convince corporate management to invest in GIS because marketing and sales data (territories) can be relatively easily overlain onto environmental data.

Finally, some packages that are add-ons to spreadsheet programs tend not to be powerful or versatile enough for use in environmental GIS work. These software packages may be valuable as an introduction to GIS techniques, however.

Data Availability and Format

After compiling the hardware and software into GIS, the next step entails accessing data layers such as:

- State and county boundaries
- Land use covers
- Water boundaries

Currently, USGS DLGs for hydrography, land use, transportation, and cultural features are available for minimal costs. Shareware programs can convert the USGS DLG3 formats into DXF (data transfer files) for import to GIS packages. These data require conversion to DXF or ARC coverage type formats for use in either ARC/INFO or MAPINFO.

The USDA Soil Conservation Service produces digital data for soil types (STATSGO and SSURGO) that users can import to ARC/INFO relatively easily. The STATSGO data cost approximately $1,000 per state and are available for most states. The detailed soil series maps, SSURGO, cost approximately $500 per county and are not available for every county in the United States. Many data layers are available for direct use by GRASS. As of yet, however, no convenient conversion utilities exist to move GRASS data to ARC/INFO or MAPINFO. The

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U.S. Fish and Wildlife Service now distributes data layers from the National Wetlands Inventory on the Internet (enterprise.nwi.fws.gov).

Other data sources available through private vendors are listed in the MAPINFO and ARC/INFO user guides and in any issue of *GIS World*. The user should be prepared to absorb significant costs if purchasing all the required data layers.

Conclusion

This study shows that the technology and software exist for a water resource manager to couple pesticide transport modeling with GIS to identify areas or individual soils that may contribute to nonpoint source pollution of water resources. The study used PC-based computing system and software. Soils maps and hydrographic maps can be easily digitized for limited cost. A skilled scientist or technician, without being a GIS expert, can run GIS to answer specific questions. The technologies this study demonstrated may be extremely valuable to managers responsible for producing SMPs.

The pesticide transport modeling performed during this study was intended for illustrative purposes. More detailed analyses, and additional simulations, would be necessary to use these data for regulatory actions or land use management. The study did, however, succeed in identifying the Manor Soil Series, with slopes exceeding 6 percent, within the Big Beaver Creek subbasin of the Pequa Creek basin, as a potential source for atrazine in runoff.

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R0021784

Integration of GIS With the Agricultural Nonpoint Source Pollution Model: The Effect of Resolution and Soils Data Sources on Model Input and Output

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Abstract

The assessment of agricultural nonpoint source pollution has been facilitated by linking data contained in a geographic information system (GIS) with hydrologic models. One such model is the Agricultural Nonpoint Source (AGNPS) Pollution Model, which simulates runoff, nutrients, and sediment from agricultural watersheds. Vector-based (ARC/INFO) and raster-based (IDRISI) GIS systems were used to generate AGNPS input parameters.

The objectives of this project were to generate AGNPS input parameters in GIS format from GIS data at different resolutions and different levels of detail (soil survey soils data versus soils data currently available in digital format from the United States Department of Agriculture). Differences in the AGNPS model sediment output based on the variations in GIS-generated AGNPS model input were evaluated.

The study also evaluated the influence of cell size resolution and soils data on sediment generated within each cell in the watershed (SGW), sediment yield from each cell in the watershed (SY), sediment yield at the watershed outlet, and peak flow. Model output was validated by comparison with measured values at the watershed outlet for a monitored storm event. Results of this study indicate that the use of different resolution GIS data and different soils data sources to assemble AGNPS input parameters affects AGNPS model output. Higher resolution data do not necessarily provide better results. Such comparisons could affect decision-making regarding the level and type of data analysis necessary to generate sufficient information.

Introduction

Agricultural runoff is a major contributor to nonpoint source pollution. Fifty-seven percent of the pollution in impaired lakes and 64 percent of the pollution in impaired rivers of the United States can be attributed to agricultural nonpoint source pollution (1). Sediment is one of the most common agricultural nonpoint source pollutants and is the largest pollutant by volume in the United States (2). More than 3 billion tons of sediment enter surface waters of the United States each year as a result of agricultural practices (1).

Accurate assessment of the effects of agricultural activities on water quality within a watershed is vital for responsible watershed management and depends on our ability to quantify the spatial variability of the watershed and the complex interactions of hydrologic processes (3). Computer models have been developed to simulate these hydrologic processes to provide estimates of nonpoint source pollutant loads. Adequate simulation of a watershed's spatial variability helps provide the best representation of hydrologic processes within the watershed.

Preservation of spatial variability within hydrologic models can be accomplished using a distributed parameter model. The distributed parameter model is more advantageous than lumped parameter models, which generalize watershed characteristics, because distributed parameter models provide more accurate simulations of the systems they model (4). One of these models is the Agricultural Nonpoint Source (AGNPS) Pollution Model. AGNPS is a distributed process model because it produces information regarding hydrologic processes at grid cells within the watershed, thus enabling preservation of the spatial variation within the watershed. Distributed parameter models integrate well with GIS because GIS can replicate the grid used in a distributed parameter model. Manual compilation of AGNPS input parameters required to evaluate small areas at low resolution (large grid cells) is relatively easy. Manually assembling data to evaluate larger areas at finer resolutions becomes tedious, however. The integration of GIS data with the AGNPS model facilitates data assembly and manipulation (5).

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Several researchers have integrated AGNPS with GIS (4-11). Smaller cell sizes within distributed parameter models are thought to best represent spatial variability within a watershed (5, 10). Certain AGNPS input parameters show sensitivity to changes in grid cell size, affecting sediment yield output (11). The use of GIS to generate input parameters for the AGNPS model enables analysis of watersheds at higher resolutions than would be practical using manual methods (5).

Research Hypotheses

The project investigated the following research hypotheses:

- AGNPS output at the highest resolution will better approximate sediment yield at the watershed outlet.
- AGNPS output for sediment generated within each cell in the watershed at highest resolution will best reflect the watershed processes.
- AGNPS output generated from the more detailed soil survey data will better estimate watershed processes.

The project also investigated other questions: will certain AGNPS input parameters (cell land slope, soil erodibility [K], the cropping factor [C], and the U.S. Department of Agriculture [USDA] Soil Conservation Service [SCS] curve number [CN] show sensitivity to changes in grid cell size? How does slope affect model output? Does a qualitatively significant difference exist between model input parameters and output calculated from data sets generated at different resolutions with different levels of detail in soils output?

Objectives/Tasks

The research in this project included analyses of:

- Certain AGNPS input parameters generated at different resolutions (10- x 10-, 30- x 30-, 60- x 60-, and 90- x 90-meter resolutions).
- AGNPS output for sediment yield (SY) and sediment generated within each cell (SGW) at different resolutions (center cells of 10- x 10-, 30- x 30-, 60- x 60-, and 90- x 90-meter resolutions).
- AGNPS output generated from different levels of detail in the soils input data (soil survey versus STATSGO data sources).

Significance

Version 4.03 of AGNPS was released in June 1994. Version 4.03 allows for evaluation of 32,767 cells. This version allows for cell sizes from 0.01 to 1,000 acres (approximately 6.36- x 6.36-meter resolution to 2,012- x 2,012-meter resolution). Previous versions of AGNPS limited the number of cells to 3,200 and the cell size resolution to 0.4 hectares (or 63.25 x 63.25 meters) (12). Reviewed literature provides no evidence that AGNPS has been used to evaluate a watershed at 10- x 10-meter resolution.

The soils data in this study were compiled at two different levels of detail. Soils data at the 1:20,000 soil survey level were generated in digital format. This level of detail was compared with soils data at the State Soil Geographic (STATSGO) database level with a scale of 1:250,000. Reviewed literature mentions no previous studies comparing AGNPS output with input generated from these two different levels of detail in soils input.

Technology for collecting and processing geographic data is continuously improving. Currently, the United States Geological Survey (USGS) 1:24,000 digital elevation models (DEMs) are available at 30- x 30-meter resolution. New satellite technology will enable DEM data to be available at 10- x 10-meter resolution, or higher. Certain satellites currently provide land cover data at 10- to 30-meter resolution (13).

An important objective of this project was to determine whether higher resolution data provide different results when routed through AGNPS. Does spatial data at higher resolutions provide better information? This paper describes the results of an analysis of AGNPS output based on different levels of both resolution and soils detail in GIS data input sources.

Materials

The Study Area

An ongoing effort is underway to clean up Onondaga Lake, Onondaga County, New York. To accomplish this effort, areas contributing agricultural nonpoint source pollution to the lake are being evaluated.

The Onondaga Lake watershed is approximately 287.5 square miles, with 40 subwatersheds. The subwatersheds in the agricultural portion of the Onondaga Lake watershed (south of Syracuse, New York) have been isolated for study of their potential nonpoint source contributions to Onondaga Lake (see Figure 1). The study area watershed (1.84 square miles, 1,177.5 acres) is one of these agricultural subwatersheds. GIS data were collected within the Otisco Valley quadrangle (USGS 1:2,400), which includes the southern portion of the Onondaga Lake watershed. Elevations in the study watershed range from 1,820 feet to 1,203 feet, with an average elevation of 1,510 feet. The watershed perimeter is approximately 6.5 miles (34,505 feet). The streams in the watershed flow from south to north to Rattlesnake Gulf, with a stream length of approximately 3.08 miles (16,265.4 feet). The stem fall of the main stream stem is quite steep at 283 feet per mile. The drainage density of the watershed is 1.67 miles of stream per square mile. Land use in the watershed is predominantly agricultural (82.8 percent).

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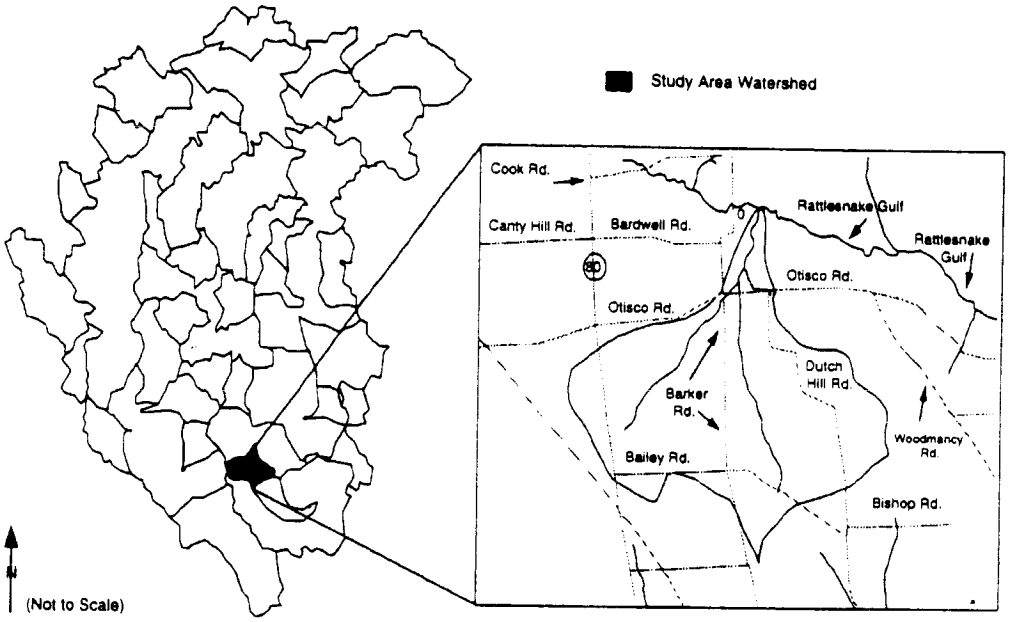

Figure 1. Onondaga Lake watershed and study area (not to scale).

The AGNPS Model

AGNPS was developed to analyze and provide estimates of runoff water quality, specifically to evaluate sediments and nutrients in runoff from agricultural watersheds for a specific storm event (11). To use AGNPS, a watershed is divided into cells of equal area. Calculations for each of the model output values are made within each cell based on the watershed data contained in each cell. Approximately 1,000 people in 46 different countries use the AGNPS model. Users include students, university professors, government agencies, lake associations, and environmental engineers.[1] AGNPS was developed in 1987 by the Agricultural Research Service (ARS) in cooperation with the Minnesota Pollution Control Agency (MPC) and the SCS. The model runs on an IBM-compatible personal computer.

Data Sources

The GIS packages of ARC/INFO Version 3.4D (14) and IDRISI Version 4.1 (15) were used to prepare input parameters for the AGNPS model. AGNPS input parameters were derived from three base maps—land use, a DEM, and soils. Table 1 shows the 22 input parameters that AGNPS required, and the base source for the data.

The land use map was obtained from a classified ERDAS image (resolution of 28 x 28 meters). The image was converted to IDRISI, brought into ARC/INFO, and regridded based on the resolution requirements of each data set. USGS could not provide a DEM for the study area, so the DEMs were interpolated from points digitized in ARC/INFO based on the Clarke method (16). The DEMs were interpolated in IDRISI on 10- x 10-, 30- x 30-, 60- x 60-, and 90- x 90-meter resolution surfaces.

Soil survey data were obtained from Onondaga County Soil Survey air photographs. An orthophoto of the 7.5 minute quadrangle was obtained from the USGS and was used with a zoom transfer scope to ortho-correct the soil survey data. The corrected soil polygons were then digitized in ARC/INFO. The Otisco Valley quadrangle comprises 79 soils mapping units. Thirty-eight different mapping units occur in the study area watershed.

The USDA SCS (now the Natural Resource Conservation Service [NARCS]) provides digital soils data from its STATSGO database. The mapping scale of STATSGO data is 1:250,000, thus it is best suited for broad planning and management uses. The number of soil polygons per quadrangle is between 100 and 400, and the minimum area mapped is 1,544 acres. The STATSGO soil data used in this project were obtained from the Onondaga County Soil Conservation Service. Approximately seven STATSGO soil groups were identified for the Otisco Valley quadrangle. Only one STATSGO soil type occurs in the study area watershed (Honeyoe silt loam).

Methods

AGNPS input parameters that showed sensitivity to changes in grid cell size in previous studies were compared between the resolutions. The AGNPS model was run eight times using precipitation values from the actual

[1] Personal communication from AGNPS Technical Support, September 1994.

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Table 1. AGNPS Input Parameters

#	AGNPS Parameter	Root Data Source	General Derivation of Input
1.	Cell number	Watershed map	Program written to determine #
2.	Cell division	Not applicable	No cell division, assumed 0
3.	Receiving cell number	Aspect map from DEM	Program written to determine #
4.	Receiving cell division	Not applicable	No cell division, assumed 0
5.	Flow direction	Aspect map from DEM	Reclassed 1-8 from azimuth map
6.	SCS curve number	Land use and soils coverage	SML written to determine CN
7.	Land slope percentage	Slope map from DEM	Provided in slope percentage from IDRISI
8.	Slope shape factor	Algorithm	Assume uniform slope
9.	Average slope length	Table of values	Obtained from SCS
10.	Manning's n coefficient	Literature values	Attached to land use database
11.	USLE K factor	SCS and soil survey	Attached to soils database
12.	USLE C factor	Literature values	Attached to land use database
13.	USLE P factor	Literature values	Attached to land use database
14.	Surface condition constant	Land use coverage	Attached to land use database
15.	Chemical oxygen demand	Land use coverage	Attached to land use database
16.	Soil texture	Soil survey	Attached to soils database
17.	Fertilizer indicator	Land use coverage	Assumed for agricultural land class
18.	Pesticide indicator	Land use coverage	Assumed for agricultural land class
19.	Point source indicator	USGS 1:24,000 map	Points in ARC/INFO and IDRISI
20.	Additional erosion	Field survey, known gullies	Assume no additional erosion
21.	Impoundment indicator	1:24,000 map, field survey	Assume no impoundments
22.	Channel indicator	Streams coverage	Assume no significant channel

storm that was monitored. Each time, the model was run using an input file created with the different input data sources as follows:

1. 30- x 30-meter resolution-soil survey data.
2. 30- x 30-meter resolution-STATSGO data.
3. 60- x 60-meter resolution-soil survey data.
4. 60- x 60-meter resolution-STATSGO data.
5. 90- x 90-meter resolution-soil survey data.
6. 90- x 90-meter resolution-STATSGO data.
7. Center cells of the 10- x 10-meter resolution—soil survey data.
8. Center cells of the 10- x 10-meter resolution—STASGO data.

As grid cell size increases, the time required to assemble data as well as the space required to store the data files increase. If a cell size resolution is cut in half, the number of cells in that coverage quadruples. In the study area watershed, increasing grid cell size from 90- x 90-meter resolution to 60- x 60-meter resolution created 778 more cells within the watershed. Moving from 60- x 60-meter resolution to 30- x 30-meter resolution added 3,646 cells to the watershed, and moving from 30- x 30-meter resolution to 10- x 10-meter resolution added 40,115 cells to the watershed data set (see Figure 2). Due to the 32,767-cell limitation of AGNPS Version 4.03, AGNPS output for SY and SGW at the 10- x 10-meter resolution (which contains 45,104 cells) could not be obtained. Input parameters at 10- x 10-meter resolution, however, could be compared with input parameters at 30- x 30-meter resolution.

Methodology of Data Analysis

The input parameter "maps" were converted to IDRISI files and combined in a format that could be routed through the AGNPS model. AGNPS model output for soil generated within each cell and for sediment yield was assembled. The 30- x 30-meter resolution maps were compared with the 60- x 60-meter resolution maps; the 60- x 60-meter resolution maps were compared with the 90- x 90-meter resolution maps; and the 30- x 30-meter resolution maps were compared with the 10- x 10-meter resolution maps.

A method for comparing maps with different grid sizes was developed so that maps of different resolutions

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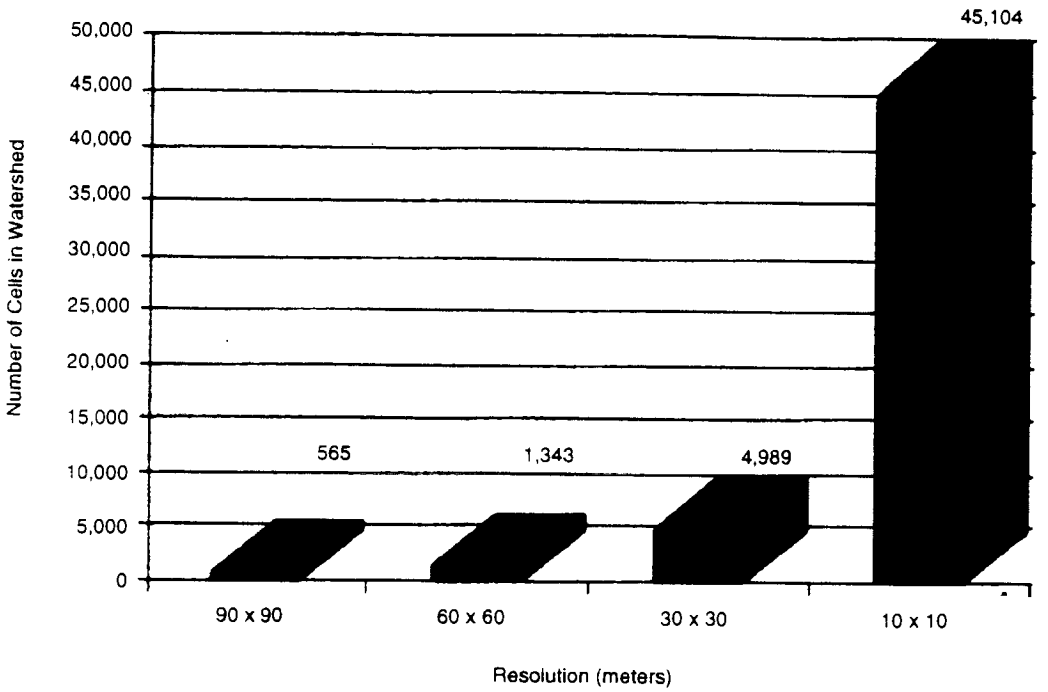

Figure 2. Effect of resolution on study area data set.

could be compared using the root mean square (RMS) statistic. This method was selected to provide a means for statistical analysis that could accommodate the spatial variability of this data set. The 60- x 60-meter resolution maps were "expanded" in IDRISI. This duplicated each grid cell so that the original 60- x 60-meter resolution map was equivalent to the 30- x 30-meter resolution map. The IDRISI command "RESAMPLE" was used to bring the 60- x 60-meter resolution map data onto the same grid size as the 90- x 90-meter resolution map. The center cells of the 10- x 10-meter resolution map were selected for comparison with the 30- x 30-meter grid cells. This comparison is based on the fact that the center cell of the 10-meter resolution is the cell that best corresponds to the entire cell of 30-meter resolution (see Figure 3).

Once maps were registered on comparable grids, the RMS difference between the maps of differing resolution was used to compare the difference between the maps

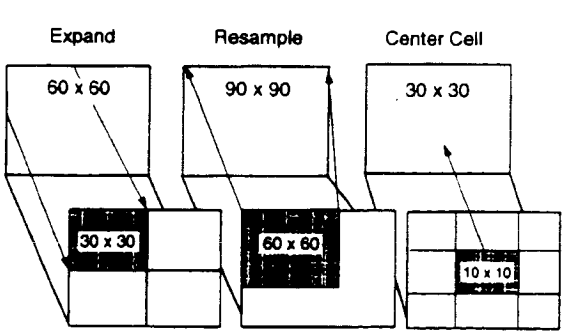

Figure 3. Method for comparing cells of differing resolution.

and the effect of cell size resolution. The RMS statistic is a measure of the variability of measurements about their true values. The RMS is estimated by comparing values in one grid system with the values in the comparison grid system. The difference between corresponding values in each grid system is squared and summed. The sum is then divided by the number of measurements in the sample to obtain a mean square deviation. Finally, the square root of the mean square deviation is calculated. The RMS difference quantifies the discrepancy between two data sets.

$$\text{RMS} = \sqrt{\sum_{i=1}^{n} \frac{(\text{grid1} - \text{grid 2})^2}{n}}$$

Results

The Storm Event

The storm that was monitored for the purposes of this field validation occurred on August 28, 1994, at approximately 8:45 p.m.; the storm duration was approximately 1.5 hours. It was a high intensity, short duration thunderstorm (see Figure 4). A global flow probe (Model FP101) was used to obtain discharge velocity measurements in feet per second. These values were then multiplied to obtain cubic feet per second. The peak discharge occurred at 11:30 p.m. on August 28, 1994, with a flow discharge of 11.15 cubic feet per second. The average runoff for the period was 4.458 cubic feet per second, or 2.42 cubic

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feet per second per square mile. The runoff volume per day was 0.09 inches.

Total sediment yield was derived from the analysis of total solids measured in field samples throughout the 24-hour storm period. The samples were processed to evaluate total suspended solids (TSS) using the vacuum filtration procedure (17). A total of 1.204 tons of suspended sediment was predicted at the watershed outlet from field data samples. A LaMotte field nutrient test kit was used to measure nitrate and phosphate concentrations in the stream. Nutrient values in this watershed for this storm event were so small (phosphorous below 0.1 parts per million and nitrogen 0.3 parts per million), they were not selected as parameters to be used in evaluating and validating the AGNPS model. The AGNPS predicted nutrient output for the storm was not measurable (0.00 parts per million). The low levels of nitrogen and phosphorous in the stream channel during the storm event can be attributed to the time of year in which the stream was monitored. At the time of field validation, agricultural activities were not operating.

Results at the Watershed Outlet: Peak Flow

The peak flow values that AGNPS calculated are largest at the highest resolution and decrease as cell size increases. The peak discharge from the watershed during the monitored storm event was 11.15 cubic feet per second. Comparisons of the AGNPS predicted peak flow and the actual field-validated peak flow showed that the 30- x 30-meter resolution cells best approximate the peak flow of the watershed for the sampled storm event. As grid cells increase from 30 x 30 to 60 x 60 and from 60 x 60 to 90 x 90, the peak flow is underestimated. As grid cells decrease from 30 x 30 to 10 x 10, the peak flow is grossly overestimated (see Table 2).

Results at the Watershed Outlet: Sediment Yield

Sediment yield at the watershed outlet was determined to be 1.204 tons. In all of the resolutions, the amount of sediment deposited at the watershed outlet cell increased as cell resolution increased (see Table 2). For this particular watershed in this particular storm, the AGNPS model overestimated the sediment yield predicted at the watershed outlet at the 10- x 10-, 30- x 30-, and 60- x 60-meter resolutions and underestimated sediment yield at the 90- x 90-meter resolution. Table 2 includes the information that AGNPS predicted for the cell designated as the watershed outlet within each resolution. (The results reported include output from the center cells of the 10- x 10-meter resolution data set, routed through the AGNPS model. Although these values are reported, the results from this data set cannot be assumed to approximate the sediment output that would result had the entire 10- x 10-meter resolution data set been simulated.)

Soil Survey Versus STATSGO Data

The Kappa statistic (14,18) was used as an indicator of similarity to describe the differences between the AGNPS output for SY and SGW generated from STATSGO and soil survey data. Results (see Table 3) indicate that no significant difference exists between the output derived from the STATSGO and soil survey data inputs within

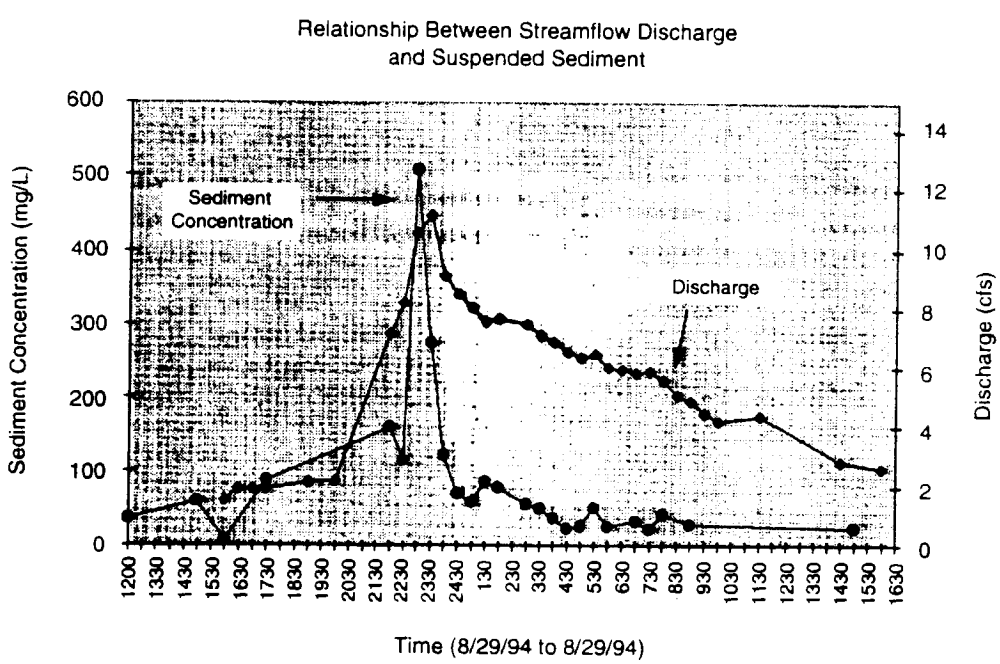

Figure 4. Storm event hydrograph and pollutograph.

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Table 2. AGNPS Results at the Watershed Outlet Versus Actual Field Values

Resolution (meters)	Predicted Soil Survey Data	Difference From Actual	Predicted STATSGO Data	Difference From Actual
10 x 10 (center cells)				
Peak runoff rate (cfs)[a]	29.88	+18.73	29.88	+18.73
Total sediment yield (tons)	4.48	+3.49	4.69	+3.28
30 x 30				
Peak runoff rate (cfs)	11.27	+0.12	11.27	+0.12
Total sediment yield (tons)	2.84	+1.64	3.11	+1.91
60 x 60				
Peak runoff rate (cfs)	9.96	-1.19	9.96	-1.19
Total sediment yield (tons)	2.11	+0.91	2.12	+0.92
90 x 90				
Peak runoff rate (cfs)	8.87	-2.28	8.87	-2.28
Total sediment yield (tons)	0.86	-0.34	0.87	-0.33

[a]cfs: cubic feet per second.

Table 3. Kappa Coefficient of Similarity

Resolution (meters)	Kappa
10 x 10 (center)	
Soil survey versus STATSGO SY	0.9866
Soil survey versus STATSGO SGW	0.9703
30 x 30	
Soil survey versus STATSGO SY	0.9859
Soil survey versus STATSGO SGW	0.9785
60 x 60	
Soil survey versus STATSGO SY	0.8960
Soil survey versus STATSGO SGW	0.7542
90 x 90	
Soil survey versus STATSGO SY	0.8743
Soil survey versus STATSGO SGW	0.6574

the same resolutions. This may be due to the homogeneity of the soil textures in both soils data sets (both dominated by silty soils).

Effects of Resolution on SGW and SY

AGNPS output for SGW and SY was evaluated for every cell within the watershed. The RMS difference was applied to determine the relative effect of input data resolution on SY and SGW output (see Table 4 and Figure 5).

Table 4. RMS for AGNPS Sediment Loss Output

Description	SGW Pounds	SY Pounds
Soil Survey Data Constant		
10 centers to 30-meter resolution	168.38	344.22
30- to 60-meter resolution	125.43	739.75
60- to 90-meter resolution	93.50	498.01
30- to 90-meter resolution	29.91	711.49
STATSGO Data Constant		
10 centers to 30-meter resolution	164.45	312.48
30- to 60-meter resolution	125.37	782.07
60- to 90-meter resolution	97.07	501.81
30- to 90-meter resolution	122.04	747.51

Moving to higher cell resolutions increasingly affects sediment generated within each cell; the largest difference in sediment generated within each cell occurs as cell resolution increases from 30 x 30 to 10 x 10 meters. Sediment generated within each cell is least affected by moving from 60 x 60 to 90 x 90 meters. Sediment yield per cell is most affected when cell resolution increases from 60 x 60 to 30 x 30 meters and least affected by increasing resolution from 30 x 30 to 10 x 10 meters.

These results prompted an assessment of the methods used to compare resolutions, to determine whether the effect on sediment yield between the 30- x 30- and 60- x 60-meter resolutions could result from the method used in comparing the resolutions (expansion of the 60- x 60-meter resolution). The procedure of comparison between the 30- x 30- and the 60- x 60-meter resolutions was repeated; however, rather than expanding the 60 x 60 data file, the 60 x 60 data file was "resampled" onto the 30- x 30-meter resolution grid, then the files were compared. The RMS results (see Table 5) show that both methods for comparing data between the resolutions provide essentially the same results. The effect of resolution on sediment yield per cell is, in fact, greatest as resolution increases from 60 x 60 to 30 x 30 meters.

Results: AGNPS Input Parameters of Concern

Previous AGNPS analyses have shown sediment yield (and sediment-associated nutrient yields) to be most affected by AGNPS inputs for cell land slope, the soil erodibility factor (K), the Universal Soil Loss Equation's

Table 5. Comparison of RMS for 30 x 30 to 60 x 60

Method	RMS SGW Pounds	RMS SY Pounds
Expansion of 60 x 60 to 30 x 30	125.43	739.75
Resampling 60 x 60 to 30 x 30	125.37	739.30
Difference	0.06	0.45

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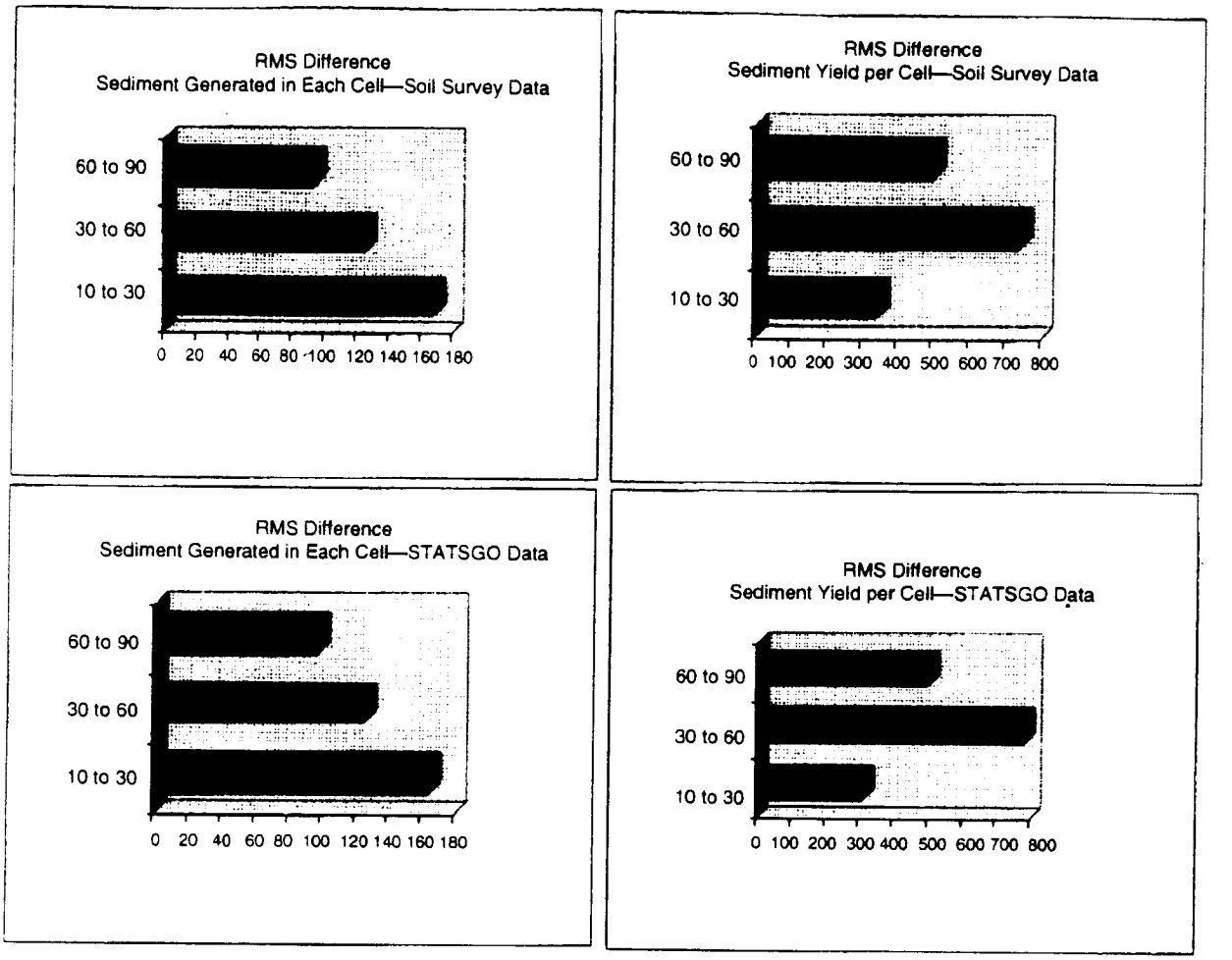

Figure 5. RMS for AGNPS sediment output.

(USLE) cropping management factor (C) and the SCS curve number (CN). To address the concerns regarding the influence of these parameters on sediment yield, the RMS differences (see Table 6 and Figure 6) and general statistics (see Table 7) for these parameters were computed.

Discussion

When evaluating the RMS as an indicator of the effect of resolution on input parameters and output sediment values, looking at the overall trend between resolutions, rather than focusing on specific values, is important. The RMS statistics for the soil erodibility factor (K), the cropping management factor (C), and the SCS curve number (CN) are least affected by a decrease in cell size resolution from 10 x 10 meters to 30 x 30 meters. These parameters are most affected when cell size resolution decreases from 30- x 30-meter to 60- x 60-meter resolution. As resolution decreases further from 60 x 60 meters to 90 x 90 meters, the effect on RMS decreases. The small-large-smaller trend in the RMS for these parameters is the same trend seen in the RMS for sediment yield throughout the watershed. The sediment

Table 6. RMS Difference: AGNPS Input Parameters of Concern

Parameters of Concern	10 to 30	30 to 60	60 to 90
K value	0.0058	0.054	0.051
Cropping factor	0.03	0.173	0.015
SCS curve number	2.11	11.40	8.40
Slope	8.17	5.59	3.68

yield within each cell therefore seems to be most affected by these input parameters. The general statistics for each of these parameters of concern show that very little difference exists in the values within each resolution, with the exception of slope. Slope values are higher at the higher resolutions and decrease as resolution increases. This is related to the method in which the GIS calculates slope.

The RMS statistics comparing resolutions for sediment generated within each cell follow the same trend as the RMS statistics for slope percentage. As resolution increases, so do the discrepancies between the compared

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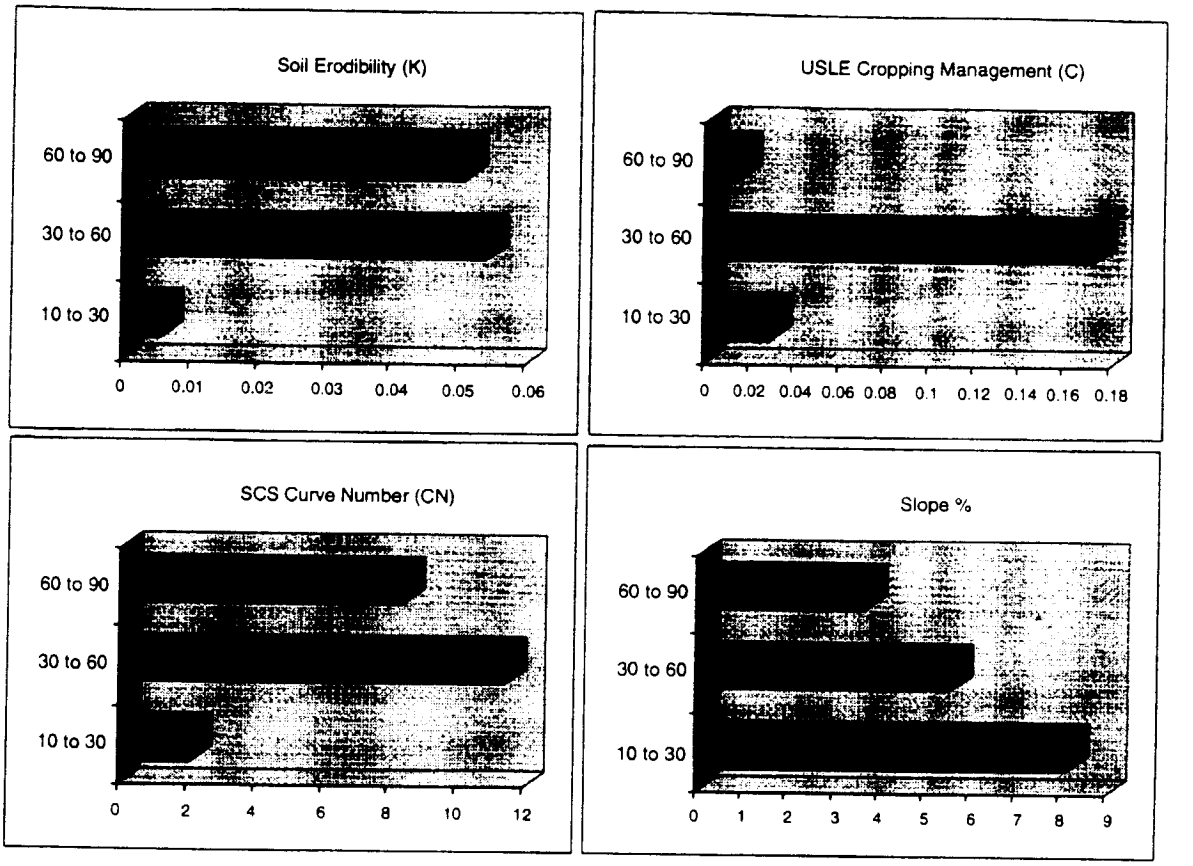

Figure 6. RMS difference for AGNPS input parameter of concern.

data sets. This trend between resolutions indicates that slope values influence sediment generated within each cell in the watershed.

The results from the 10- x 10-meter resolution data set were obtained by selecting the center cells of the 10- x 10-meter resolution data set and routing the data from this set through AGNPS using the flow pathways developed for the 30- x 30-meter resolution. The results do not provide the same information as would be provided had the entire 10- x 10-meter data set been routed through AGNPS. The RMS values obtained from comparisons of the 10- x 10-meter resolution input parameters with the 30- x 30-meter resolution input data reveal that little difference exists between the data in these resolutions. Comparison of the center cell 10- x 10-meter data set output with field monitored data shows that the 10 x 10 center cell data overestimates both peak flow and sediment yielded at the watershed outlet. This can be attributed to the larger slope values in this resolution.

Conclusion

This study used GIS to generate data files for application to the AGNPS model. The objectives of this project were to evaluate the effect of different levels of detail used in generating the input files on selected input and output parameters. The results show that, for a watershed with characteristics equivalent to those of the study area watershed, differences exist in model output based on the cell size resolution of the watershed.

The selected cell size resolution directly affects slope values. The influence of the slope parameter dominates AGNPS predictions for sediment generated within each cell and sediment yield at the watershed outlet in the study area watershed. The indicated parameters of concern have the most influence on sediment yield for each cell in the watershed. The greatest variation in the indicated parameters of concern and thus the sediment yield output occurs between the 30- x 30-meter and 60- x 60-meter resolutions. AGNPS estimates for sediment yield in files generated from STATSGO data were larger than sediment yields from files generated with soil survey soils data in the 30- x 30-, 60- x 60-, and 90- x 90-meter resolutions. For this watershed, however, no significant difference existed between data generated from soil survey and STATSGO data sources as indicated by the kappa coefficient of similarity.

Results predicted by the AGNPS model at the watershed outlet were compared with results from an actual storm monitored at the watershed outlet. The 30- x 30-meter resolution data set provided the most accurate

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Table 7. Statistics for AGNPS Input Parameters of Concern

Value	10 x 10	30 x 30	60 x 60	90 x 90
K Value (units of K)				
Average	0.2989	0.2989	0.2882	0.2889
Maximum	0.49	0.49	0.37	0.49
Minimum	0.17	0.17	0.17	0.17
Standard deviation	0.0453	0.0453	0.0513	0.0456
C Factor (units of C)				
Average	0.0306	0.0306	0.0295	0.0295
Maximum	0.076	0.076	0.076	0.076
Minimum	0	0	0	0
Standard deviation	0.0212	0.0211	0.0213	0.0208
CN (units of CN)				
Average	71.004	71.003	70.75	70.68
Maximum	100	100	100	100
Minimum	55	55	55	55
Standard deviation	9.13	9.08	8.97	8.84
Slope (%)				
Average	34.13	30.153	27.67	26.38
Maximum	567	224	152	99
Minimum	0	0	0	0
Standard deviation	33.75	23.45	18.97	16.44

prediction for peak flow at the watershed outlet. AGNPS output in the 10- x 10-meter center, 30- x 30-meter, and 60- x 60-meter resolutions overestimated the actual sediment yield recorded at the watershed outlet for the validated storm event.

For the study area watershed, cell size resolution of 30 x 30 meters seems appropriate based on the accurate AGNPS model prediction for peak flow when validated with the field-monitored storm. The steep slopes created in the 10- x 10-meter resolution data set may lead to an overestimation of sediment output, rendering data at this resolution unreliable. At this time, the 10- x 10-meter resolution is both impractical and infeasible for use with the AGNPS model.

AGNPS output at the highest resolution does not provide a better approximation of sediment yield at the watershed outlet. AGNPS output for sediment generated within each cell in the watershed at the highest resolution does not accurately simulate the watershed processes. AGNPS output generated from the more detailed soil survey data is not significantly different from data generated by the STATSGO digital soils database.

This study raises the following questions:

- What level of detail is both practical and acceptable for policy-making and decision-making?
- What constitutes a cost-effective analysis?

Ultimately, these questions are best answered on a case-by-case basis and should be determined based on the size of the study area and on how the results of the analysis will be used (i.e., to make a direct land use decision or for broader planning). For broad planning analyses on large watersheds, the benefit of digitizing the soil survey data is outweighed by the cost in time and effort to generate this detailed database. STATSGO data may be sufficient. If a direct land management decision is being made for a small area such as a farm within a watershed, however, the analysis should use the most detailed soils data.

Recommendations for Future Work

The original intent of this study was to use the capabilities of AGNPS Version 4.03 to evaluate a watershed using data generated at a high cell size resolution—10 x 10 meters. AGNPS h

ad never been used to evaluate data at such a high resolution. As discovered during this project, the newest version of AGNPS is not, at this time, capable of handling a data set that has more than 32,767 cells (19). Once this limitation with the AGNPS model is remedied, the entire 10- x 10-meter resolution data set should be routed through the model so that definite conclusions regarding the applicability of such a detailed data set can be made.

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XGRCWP, a Knowledge- and GIS-Based System for Selection, Evaluation, and Design of Water Quality Control Practices in Agricultural Watersheds

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Abstract

The Expert GIS Rural Clean Water Program (XGRCWP) integrates a geographic information system (GIS), a relational database, simulation models, and hypertext mark language documents to form an advisory system that selects, evaluates, sites, and designs nonpoint source pollution control systems in agricultural watersheds. Its major features include:

- Customized GIS functions to obtain spatial and attribute data and feed them to a rule-based expert system for selecting feasible control practices.
- A user interface for examining the field-specific conditions and recommended control practices on the screen by clicking on the displayed field boundary map.
- A direct linkage between the GIS spatial data and the relational attribute data, which allows users to examine data on the screen interactively.
- A graphic user interface to GIS functions, which enables users to perform routine watershed analyses.
- Linkage to hypertext reference modules viewable by Mosaic Internet document browser.
- Dynamic access to other models such as the Agricultural Nonpoint Source Simulation Model.

The software environment of XGRCWP is GRASS 4.1 and X-Windows on SUN OS 4.3.1. Its major functions have been tested for the Sycamore Creek watershed in Ingham County, Michigan.

Introduction

In 1981, the U.S. Environmental Protection Agency (EPA) and the U.S. Department of Agriculture (USDA) initiated the Rural Clean Water Program (RCWP) in 21 agricultural watersheds. This program represents the most intensive water quality monitoring and implementation and evaluation of nutrient, sediment, and pesticide reduction practices ever undertaken in the United States (1). More than a decade of research efforts has resulted in a wealth of experiences and lessons on selection, siting, and evaluation of nonpoint source control practices.

The storehouse of knowledge gained from RCWP is of little use, however, unless it is properly integrated and packaged in an easily accessible form. Technology transfer of this knowledge is therefore critically important. To integrate and synthesize the lessons learned from RCWP, Penn State University initiated an RCWP expert project. The hypertext-based version of the RCWP expert system, completed in 1993, can select and evaluate nonpoint source control systems at a single site. Although the hypertext-based version is still suitable for users who do not have access to geographic information systems (GIS) data, it is inadequate for the comprehensive selection and evaluation of control systems on a watershed basis. It does not provide the user the spatial reference of a site and requires the user's subjective judgment for the model input.

The Expert GIS Rural Clean Water Program (XGRCWP) is the UNIX and X-Window version of the RCWP expert system, which integrates GIS and the RCWP expert system to provide decision support at multiple spatial scales from single fields to subwatersheds to the watershed scale. This paper presents the major features of XGRCWP, including design of the expert system, interface to GIS functions, and linkages to a relational database and simulation models.

Overview

XGRCWP comprises five major components (see Figure 1):

- An expert system for recommending control practices based on site-specific information.

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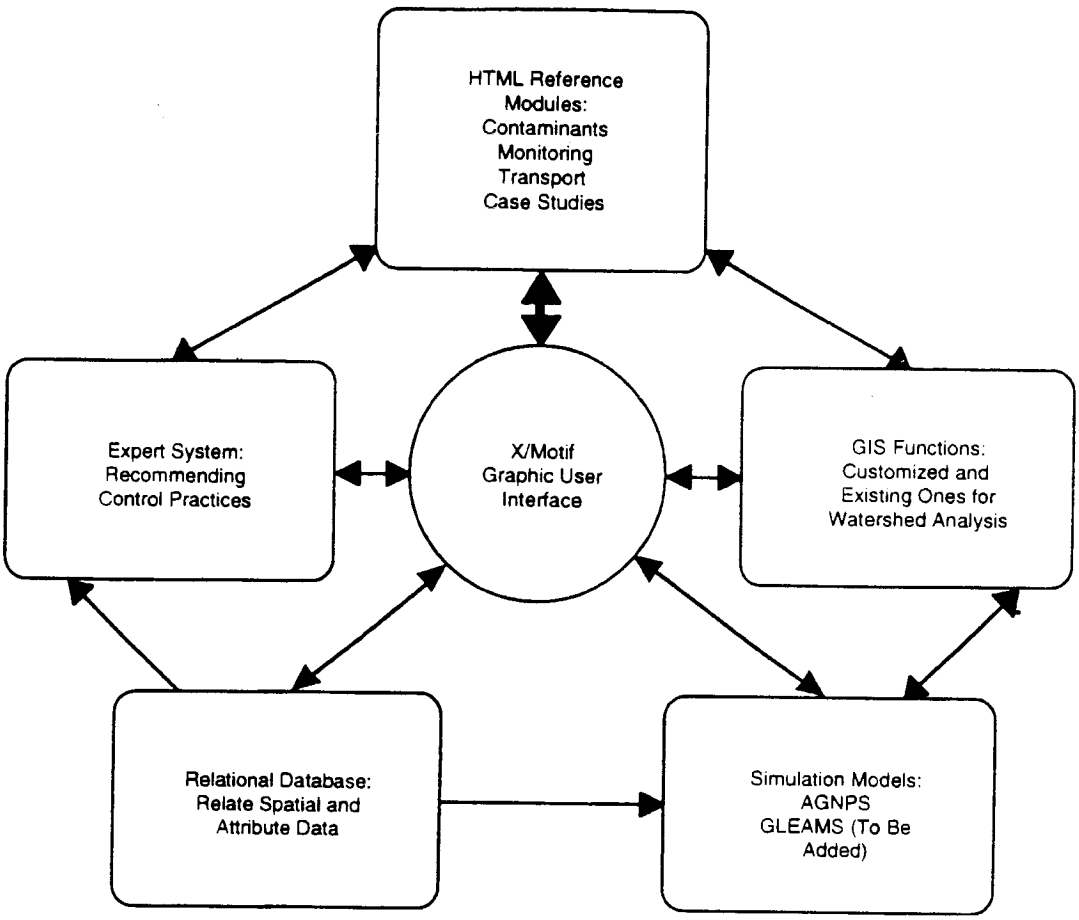

Figure 1. Major components of XGRCWP and their relationships.

- Custom and existing GIS functions for watershed analysis and estimation of contaminant loading potential.
- Linkage to fields, soils, and land use databases.
- Linkage to the Agricultural Nonpoint Source Simulation Model (AGNPS) (2).
- Hypertext mark language (HTML) reference modules.

The X/Motif graphic user interface (GUI) integrates the five components and allows the user to navigate flexibly among them. The components are also internally connected in different ways. For example, the expert system can use the customized GIS functions to retrieve site-specific information from Geographical Resource Analysis Support Systems (GRASS) (3) data layers and INFORMIX relational database tables. In addition, the expert recommendations of control practices can be displayed and examined using GRASS functions. Finally, the GIS functions can help generate input to the AGNPS model, and its output can be converted to GIS format for additional analyses.

Design of the Expert System

The objective of the expert system is to recommend feasible control systems (i.e., complementary sets of control practices to reduce nonpoint source pollution based on site-specific conditions). One distinct feature of this system is the combination of two modes of data acquisition: direct user input and GIS functions. XGRCWP also has two modes for deriving the expert recommendations: batch or interactive. This section discusses these aspects of the expert system as well as its knowledge base.

Rules for Control Practice Selection

The knowledge base of the expert system includes the following six site-specific characteristics:

- Contaminant of interest and its adsorption characteristic.
- Potential level of contaminant loading (low, medium, or high).
- Potential level of contaminant leaching (low, medium, or high).
- Soil hydrologic group (A, B, C, or D).

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- Time of year (during or outside the growing season).
- Type of land use (cropland, animal waste, or critical area).

The user first chooses a contaminant of interest from a list consisting of four kinds of pesticides (strongly, moderately, or weakly adsorbed, and nonadsorbed) and eight other contaminants (ammonia, bacteria, sediment, total nitrogen, total phosphorus, nitrate, orthophosphorus, and viruses). The values of other characteristics, some of which vary with the contaminant specified, can then be input either directly by the user or by custom GRASS functions as discussed in the section of this paper on data acquisition.

The RCWP used 14 general categories of control practices (see Table 1). Many suitable conditions were established for each general category. For example, conservation tillage is recommended to reduce runoff for cropland under conditions otherwise favoring loss through sediment transport, such as a contaminant strongly adsorbed to the soil (e.g., total phosphorus), the nongrowing season, and soils with a relatively high runoff potential (soil group C or D) (see Figure 2). Each general category includes several specific control practices. When a general practice category is recommended, the user must decide which specific practice within that general category to evaluate further by consulting the nonpoint source database (NPSDB) for the reported research data about this practice or by running the AGNPS simulation model.

Data Acquisition

The expert system recommends one or more control systems based on site-specific conditions that are either directly input by the user or calculated by the customized GRASS functions. The user always specifies the contaminant of interest and the season, while a GRASS function (R.HYDRO-GRP) always determines the soil hydrologic group of each field. For the other factors (loading potential, leaching potential, and application class), however, the user has two alternative ways to decide input values. For example, after the user selects a contaminant of interest, the program displays the contaminant loading potential window (see Figure 3). The potential level of the selected contaminant can be indicated if the user knows it. Otherwise, the user can let the GRASS functions derive loading potential from existing field data.

Table 1. The Best Management Practices Used in the Rural Clean Water Program

Source control practices	Nutrient Management (NUTR) Pesticide Management (PEST)
Structural control practices	Animal waste systems (AWS) Diversion systems (DIV) Sediment retention and water control (SED) Terrace systems (TERR) Waterway systems (WATW)
Vegetative control practices	Conservation tillage (CT) Critical area treatment (CAT) Cropland protection systems (CPS) Grazing land protection (GLP) Permanent vegetative cover (PVC) Stream protection (SP) Stripcropping (SCR)

The direct input option can also be used to help the user address "what-if" questions. When the user selects the GIS functions to determine the loading potential, XGRCWP makes a series of calls to appropriate customized GRASS functions according to the current contaminant of interest. For example, if the contaminant is total nitrogen, the functions R.MANURE, R.FERT, and R.B.CONCENTRATION are called to estimate total nitrogen from manure, fertilizer, and soil base concentration, respectively. Another GRASS function, R.NP.LOADING, is then called to translate the quantitative measure of loading potential into the qualitative classification (low, medium, or high) as input to the expert systems. These GRASS functions generate the inputs by searching and converting the data from INFORMIX relational data tables that are associated with the GRASS spatially referenced data, such as field boundary and soil map. Table 2 lists the customized GRASS functions developed for data acquisition.

Control System Recommendation

XGRCWP derives the expert recommendations for control systems in two ways: in a batch mode for every field in a watershed and in an interactive mode for a user-specified field.

In batch mode, an existing GRASS function, R.INFER, is used to create a raster data layer for each general practice category of control practice according to a ruleset prepared for that general category. For example, the contents and formats for the conservation tillage practice are documented in Table 3. The raster data layer for representing the conservation tillage recommendations (CT.rec) is generated by running R.INFER with the appropriate rule. The category value of CT.rec is 1 at each point in the data layer where the conservation tillage is recommended, or 0 otherwise. The R.INFER function is similarly called for other general practice categories. Additional GRASS functions can then display or further analyze the resulting map layers. The batch mode provides the user the overall picture with a watershed-wide view of feasible control systems.

In the interactive mode, the field boundary map is displayed and the user can specify any field of interest by clicking the mouse on it. The recommendations and the site-specific conditions of the field are displayed on the right half of the screen. The recommended control practices are also displayed in a popup window for further

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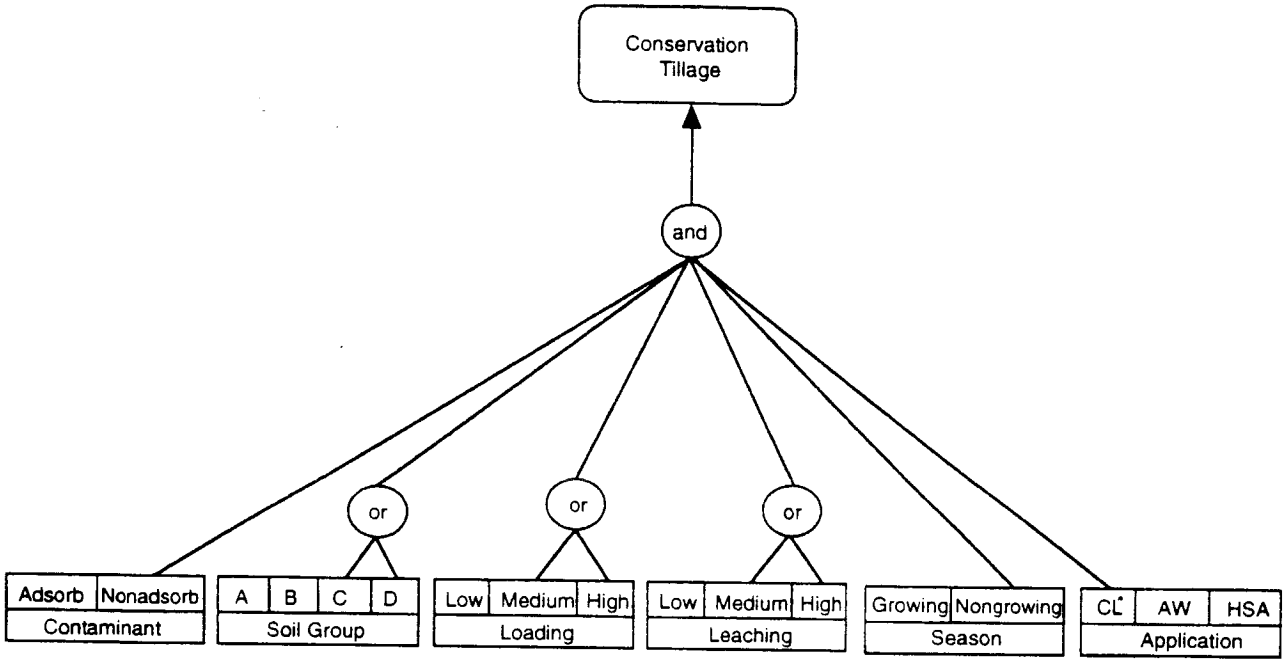

Figure 2. Dependency network (AND-OR diagram) for site-specific recommendation of conservation tillage.

examination, such as the specific practices within each general category, the feasible control systems for non-point source pollution control, and research data on the practices. The interactive mode is implemented through the integration of a Bourne shell script, structured query language (SQL) commands, a customized GRASS function (R.RCWP.EXPERT), and GRASS display functions with the Motif GUI. Interactive mode is intended for detailed consideration of a specific farm.

Interface to GIS Functions

XGRCWP provides a GUI to most of the customized GRASS functions and some of GRASS's existing functions (see Figure 4). This interface shields the user from complex syntax so the user can focus on the subject matter. The GUI makes it easier for the user to perform routine operations such as estimation of contaminant loading, identification of critical areas, erosion and runoff calculation, and other watershed analysis tasks. It also helps the user make full, effective use of all custom and some existing GRASS functions.

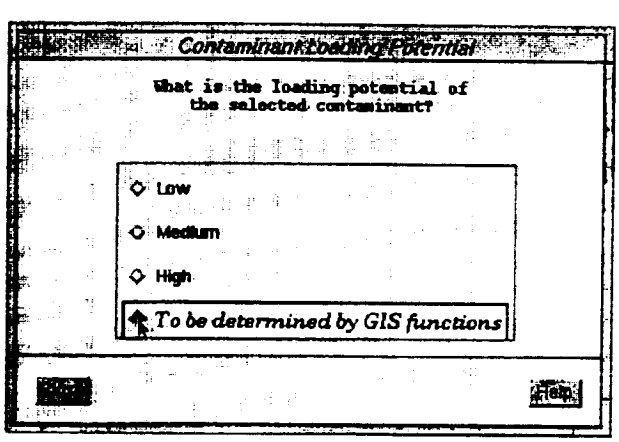

Figure 3. The popup window for the potential level of contaminant loading.

Linkages to Database and Other Models

Data Structure

The GRASS functions used to generate inputs for the expert system use the same soils and fields relational databases as the Water Quality Model/GRASS Interface under development by the Soil Conservation Service (SCS) (4). XGRCWP and our custom GRASS functions were tested for the Sycamore Creek watershed, Ingham County, Michigan. In this data structure, spatial data (e.g., field boundaries, watershed boundaries, soils map unit boundaries, and elevation data) are saved as GRASS raster data layers while attribute data (e.g., crop information, fertilizing schedule, soil information) are stored in INFORMIX relational database tables. Each field or soil map unit is assigned a unique identification (ID) number. The field attribute (INFORMIX) data also contain this ID number. The linkage between the GRASS raster map and the INFORMIX data is accomplished with a GRASS category label (see Figure 5).

Linkage to Database

To allow the interactive examination of field data from GRASS raster layers and the associated relational da-

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Table 2. Summary of the Customized GRASS Functions Developed by Nonpoint Source Agricultural Engineering Research Group at Penn State University To Generate Inputs for the RCWP Expert System

Name	Descriptions
R.FERT	Produces raster maps of total nitrogen or total phosphorous from the scheduled fertilizer applications for different crops by dynamically retrieving information from a GRASS data layer and INFORMIX data tables
R.MANURE	Calculates the total manure on each farm according to animal numbers and types (e.g., dairy cow, beef cow, horses, swine), allocates manure to the fields on a farm by a user specified strategy (uniformly spreading or inverse distance weighted distributing method), and finally estimates nitrogen and phosphorous loading from manure application rate, conversion factor, percentages of transportation losses, and volatile losses
R.B.CONCENTRATION	Estimates nitrogen and phosphorous concentration in parts per million within different types of soils according to the organic matter contents
R.NP.LOADING	Classifies the loading potential of nitrogen or phosphorous into three categories (low, medium, and high) based on the actual loading from fertilizer and manure and the N or P concentration in soils
R.EROSION	Obtains a relative measure of soil erosion severity by dividing the amount of erosion by the tolerance values of the soils and then reclassifying them into three categories (low, medium, and high)
R.LEACHING.P	Estimates leaching index from soil hydrologic group and annual and seasonal precipitation and classifies it into three categories (low, medium, and high)
R.HYDRO-GRP	Retrieves soil hydrologic group from the INFORMIX database and reclassifies the soil map into soil hydrologic groups

tabase tables, XGRCWP calls our custom function, D.WHAT.FIELD.SH, a Bourne shell script that dynamically links GRASS raster layers and the INFORMIX database tables. When the user clicks on a field, for example, this function extracts field-specific information from INFORMIX tables such as field information, fertilization schedule, crop operation schedule, and soil information. The D.WHAT.FIELD.SH function then displays all related soils and fields information for the given field. It also marks the field boundary map to remind the user which fields have already been examined.

Linkage to Reference Modules

At any stage of the selection, evaluation, siting, and design procedure for control practices, the user can consult reference modules that provide information, guidance, and data about contaminant properties, transport variables, and examples of applications from RCWP projects. Four reference modules are available in the Macintosh version of RCWP expert system: contaminants, monitoring, transport, and case studies. We are currently converting these reference modules into Mosaic-viewable HTML documents so that they can be accessed from XGRCWP. Mosaic is a public domain Internet-aware document browser that is available for X-Windows, Macintosh, and Microsoft Windows.

All four modules use graphics to demonstrate design procedures and contaminant control processes. The contaminant module provides information about 11 categories of contaminants cited in RCWP projects and their impacts on surface and ground-water resources. The monitoring module describes different aspects of water quality sampling and analysis systems. The transport module describes contaminant pathways in surface and ground water. The case studies module presents detailed examples from key RCWP projects. These examples cover both practice selection and implementation aspects of control systems. The reference modules serve as a complementary component of XGRCWP.

Linkage to AGNPS

AGNPS is a distributed-parameter, storm event-based model that estimates runoff, sedimentation, and nutrient loss in surface runoff within agricultural watersheds (2). The prototype version of the Water Quality Model/GRASS Interface developed by SCS conveniently generates an AGNPS input file for all cells in a watershed from the spatial and relational soils and fields databases. The UNIX version of AGNPS can then use this input file. XGRCWP can call AGNPS directly from its X-Window interface and convert standard AGNPS model outputs for all cells in the watershed into GRASS raster format for display and analysis.

Discussion

The literature on software systems for managing nonpoint source pollution in agricultural watersheds is diverse and rapidly growing. With few exceptions (5-7), these decision support systems are purely model-based, GIS-based (8), or hybrid systems with models running within a GIS framework (9-14). The addition of expert system components can overcome some of the difficulties in primarily model-based systems:

- Overly intensive input data requirements.
- Inability to handle missing or incomplete data.
- Requirements that all inputs be numerically expressed.

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Table 3. The Rule File for Recommending Conservation Tillage

IFNOTMAP app.class 3	!application class is not high-source area
ANDIFMAP contam.feature 2	!contaminant is nonadsorbance
ANDIFMAP leaching.p 1	!leaching potential is low
ANDIFMAP soil.g 1 2	!soil groups are A or D
ANDIFMAP contam.load 1	!contaminant loading is low
ANDIFMAP season 2	!nongrowing season
THENMAPHYP 1 yes, CT	
is recommended	
!	
IFNOTMAP app.class 3	!application class is not high-source area
ANDIFMAP contam.feature 2	!contaminant is nonadsorbance
ANDNOTMAP leaching.p 3	!leaching potential is not high
ANDNOTMAP soil.g 1	!soil groups are not A
ANDNOTMAP contam.load 3	!contaminant loading is not high
ANDIFMAP season 1	!growing season
THENMAPHYP 1 yes, CT	
is recommended	
!	
IFNOTMAP app.class 3	!application class is not high-source area
ANDIFMAP contam.feature 1	!contaminant is strong adsorbance
ANDNOTMAP contam.load 3	!contaminant loading is not high
ANDIFMAP season 1	!growing season
THENMAPHYP 1 yes, CT	
is recommended	
!	
IFNOTMAP app.class 3	!application class is not high-source area
ANDIFMAP contam.feature 1	!contaminant is strong adsorbance
ANDNOTMAP leaching.p 3	!leaching potential is not high
ANDIFMAP soil.g 3 4	!soil groups are C or D
ANDNOTMAP contam.load 3	!contaminant loading is not high
ANDIFMAP season 2	!nongrowing season
THENMAPHYP 1 yes, CT	
is recommended	

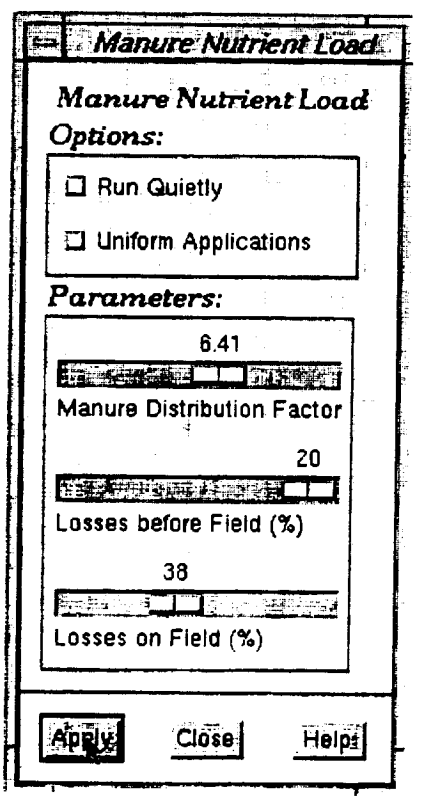

Figure 4. The GUI to the R.MANURE function.

- High degree of expertise needed to structure model input and explain model output relative to the user's problem context.

The expert system component of XGRCWP also reduces the number of model runs needed for decision support through preliminary, rule-based screening of control systems at each site of interest in the watershed.

Conclusions

XGRCWP incorporates several kinds of expertise for the user's benefit:

- Subject matter expertise in siting and selecting nonpoint source control systems in agricultural watersheds.
- Expertise in configuring AGNPS model input from the soils and fields databases.
- Expertise in interpreting, explaining, and visualizing expert system and model input.

The integration of the expert system and the GRASS GIS makes input to the expert system easier and more objective. It enhances the expert system's capability for recommending effective control practices at the field level to achieve watershed contaminant loading objectives. XGRCWP is designed as an open structured

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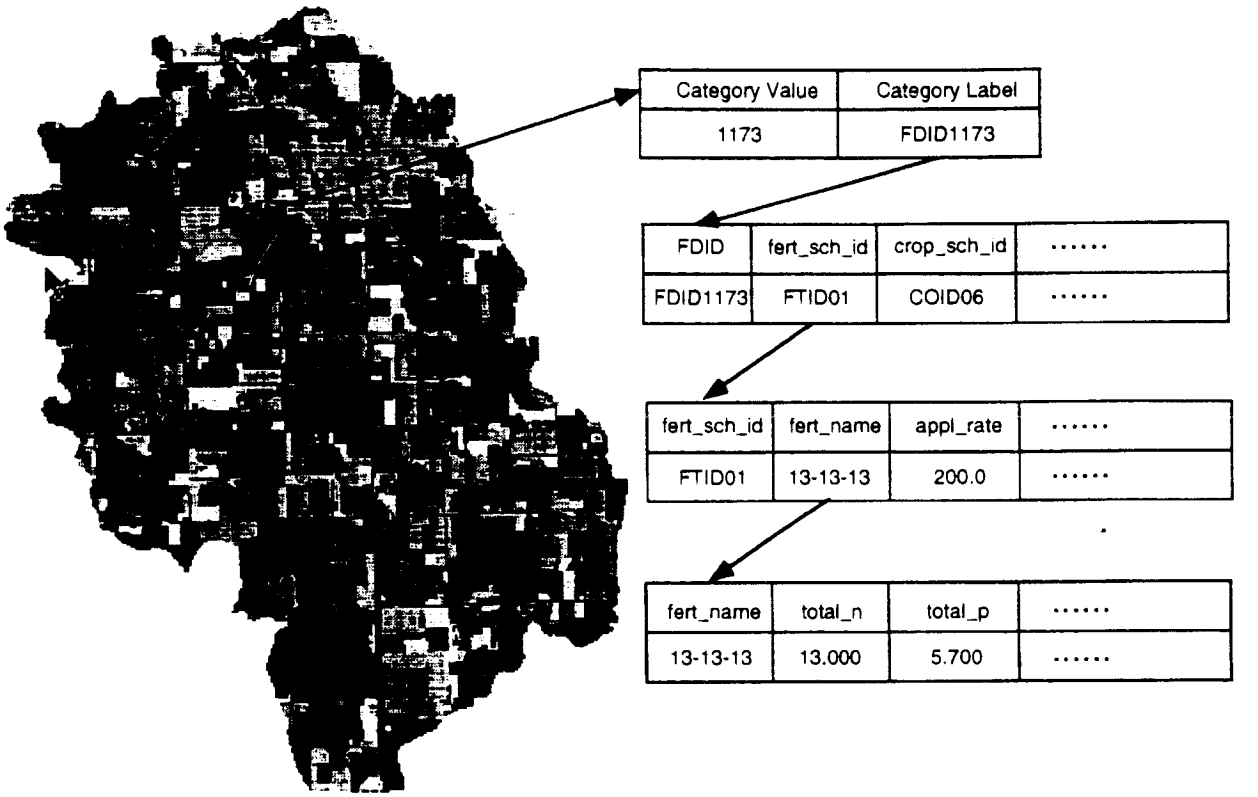

Figure 5. Data structure of Sycamore Creek watershed, Ingham County, Michigan.

program and has great potential to be improved easily and continually according to users' feedback. Ongoing efforts to enhance the program include:

- Developing more rules that incorporate topographical factors such as slope and slope length for the expert system so that more site-specific control practices can be recommended.
- Adding dynamic hypertext-based help and reference to the program.
- Establishing intelligent linkages among the expert system, the GIS functions, and other simulation or design models for nonpoint source control practices.

Acknowledgments

We thank Vicki Anderson, Ruth Shaffer, and Brent Stinson of the Michigan USDA SCS and John Suppnick of the Michigan Department of Natural Resources for their assistance in working with the underlying databases in the Sycamore Creek watershed. This project is supported by U.S. EPA Grant No. X-818240.

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R0021803

Integration of EPA Mainframe Graphics and GIS in a UNIX Workstation Environment To Solve Environmental Problems

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Abstract

The Assessment and Watershed Protection Division of the Office of Wetlands, Oceans, and Watersheds has developed water quality analysis software on the U.S. Environmental Protection Agency (EPA) mainframe computer. This software integrates national on-line environmental databases and produces maps, tables, graphics, and reports that display information such as water quality trends, discharge monitoring reports, permit limits, and design flow analyses.

In the past, this graphic software was available only to users connected to the mainframe with IBM graphics terminals or PCs with graphics emulation software. Recently, software has been developed that can be used to: 1) access the EPA mainframe from a UNIX workstation via the Internet, 2) execute the Water Quality Analysis System (WQAS) procedures, 3) display WQAS graphics in an X-Window on the workstation, and 4) download data in a geographic information system (GIS) format from the mainframe. At the same time, this workstation can execute ARC/INFO and ARC/VIEW applications in other X-Windows. This capability allows analysts to have the power of GIS, the mainframe databases (e.g., Permits Compliance System [PCS], STORET, Reach File, Industrial Facilities Discharge File, Daily Flow File, Toxic Chemical Release Inventory), and the retrieval/analysis/display software (Environmental Data Display Manager, Mapping and Data Display Manager, Reach Pollutant Assessment [RPA], PCS-STORET Interface, UNIRAS) available to them on one desktop. This capability extends the tool set available to GIS analysts for environmental problem-solving.

This paper discusses application of these tools and databases to several problems, including EPA's watershed-based approach to permitting, and the RPA, an automated method to identify priority pollutants in watersheds.

Introduction

The purpose of this paper is to explore new geographic information systems (GIS) data integration tools that are applicable to a wide range of environmental problems, including the U.S. Environmental Protection Agency's (EPA's) watershed-based approach to permitting and the Reach Pollutant Assessment (RPA), an automated method to identify priority pollutants in watersheds. The ultimate goal is to make these tools and databases accessible to a wide range of users.

Understanding aquatic resource-based water quality management depends on access to and integration of diverse information from many sources. To date, the techniques to perform this integration, and thus yield meaningful analyses supporting environmental decision-making, are neither fully developed nor documented. New tools and information resources are now available, but not used to their full potential, for more valuable water quality and watershed analyses. EPA

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headquarters is responsible for ensuring that integrated data management tools are available for water quality analyses and data reporting as well as making national data systems more useful. EPA will accomplish this by upgrading and crosslinking systems, developing interactive data retrieval and analysis mechanisms, and providing easy downloading of data to client workstations.

The Assessment and Watershed Protection Division (AWPD) of the Office of Wetlands, Oceans, and Watersheds (OWOW) has developed water quality analysis software on the EPA mainframe computer (1). This software integrates national on-line environmental databases and produces maps, tables, graphics, and reports that display information such as water quality trends, discharge monitoring reports, permit limits, and design flow analyses. In the past, this graphic software was available only to users connected to the mainframe with IBM graphics terminals or PCs with graphics emulation software. Recently, software has been developed that can be used to:

- Access the EPA mainframe from a UNIX workstation via the Internet.
- Execute the Water Quality Analysis System (WQAS) procedures.
- Display WQAS graphics in an X-Window on the workstation.
- Download data in a GIS format from the mainframe.

At the same time, this workstation can execute ARC/INFO and ARC/VIEW applications in other X-Windows. This capability allows analysts to have the power of GIS, the mainframe databases (e.g., Permits Compliance System [PCS], STORET, Reach File, Industrial Facilities Discharge File, Daily Flow File, Toxic Chemical Release Inventory), and the retrieval/analysis/display software (Environmental Data Display Manager, Mapping and Data Display Manager, RPA, PCS-STORET Interface, UNIRAS) available to them on one desktop. This extends the tool set available to GIS analysts for environmental problem-solving. This paper discusses how these tools and databases have been applied to two examples: 1) a watershed-based approach to permitting and 2) the RPA, an automated procedure for identifying watersheds with priority pollutants.

Mainframe Databases and Tools

The EPA IBM ES9000 mainframe computer, located in Research Triangle Park, North Carolina, contains a large volume of digital water quality and environmental data available on-line through a number of data retrieval and display tools (see Figure 1). Other documents describe these databases and tools in detail (2, 3).

This effort focused on showing how these databases and tools can complement GIS activities. In some cases, data can be directly downloaded to a workstation in GIS format. An example of this is accessing EPA's Reach File (Version 1) from GRIDS (4). In other cases, databases are accessed by mainframe tools, the data are processed, and a GIS data set is produced that can be downloaded to a workstation. An example of this case is the RPA (RPA3) tool that integrates data from the Reach File, STORET, and PCS to identify priority pollutants in watersheds (5).

The mainframe can be accessed through several paths: Internet, PC dialup, or dedicated line into a terminal controller (see Figure 2). In the applications presented here, the Internet connectivity is emphasized because this is the mechanism that makes these databases and tools available to GIS analysts at their workstations. Figure 3 shows the hardware and software requirements for Internet access to the water quality data integration tools. The basic components are a UNIX workstation with an X-Window manager, the X3270 software, and Internet connectivity. The X3270 software is required to emulate an IBM 3270 full-screen terminal. This software is publicly available through EPA's National Computer Center User Support. In addition, an account on the mainframe computer is required. Once this account is established, an additional software module, GDDMXD, is required to map IBM host-based graphics to the workstation's X-Window. The GDDMXD software resides on the mainframe and is loaded when the user logs in. Once the hardware and software are set up, a single UNIX workstation can provide access to mainframe and workstation tools and databases on one desktop (see Figure 4).

Applications

To illustrate how these mainframe and workstation tools/databases can work together to solve environmental problems, we present two applications. The first shows a watershed-based approach to permitting; the second describes the RPA.

Watershed-Based Approach to Permitting

The watershed approach is a process to synchronize water quality monitoring, inspections, and permitting to support water quality protection activities on a geographic basis. It is a coordinated and integrated method to link science, permits, and other pollution control and prevention activities to meet state water quality standards. Numerous local, state, and federal agencies have recognized watershed approaches as the best way to manage natural resources effectively and efficiently. Establishing a schedule for data collection, permit issuance, and other elements of this approach affords the opportunity to coordinate and integrate other natural resource management efforts to make better use of

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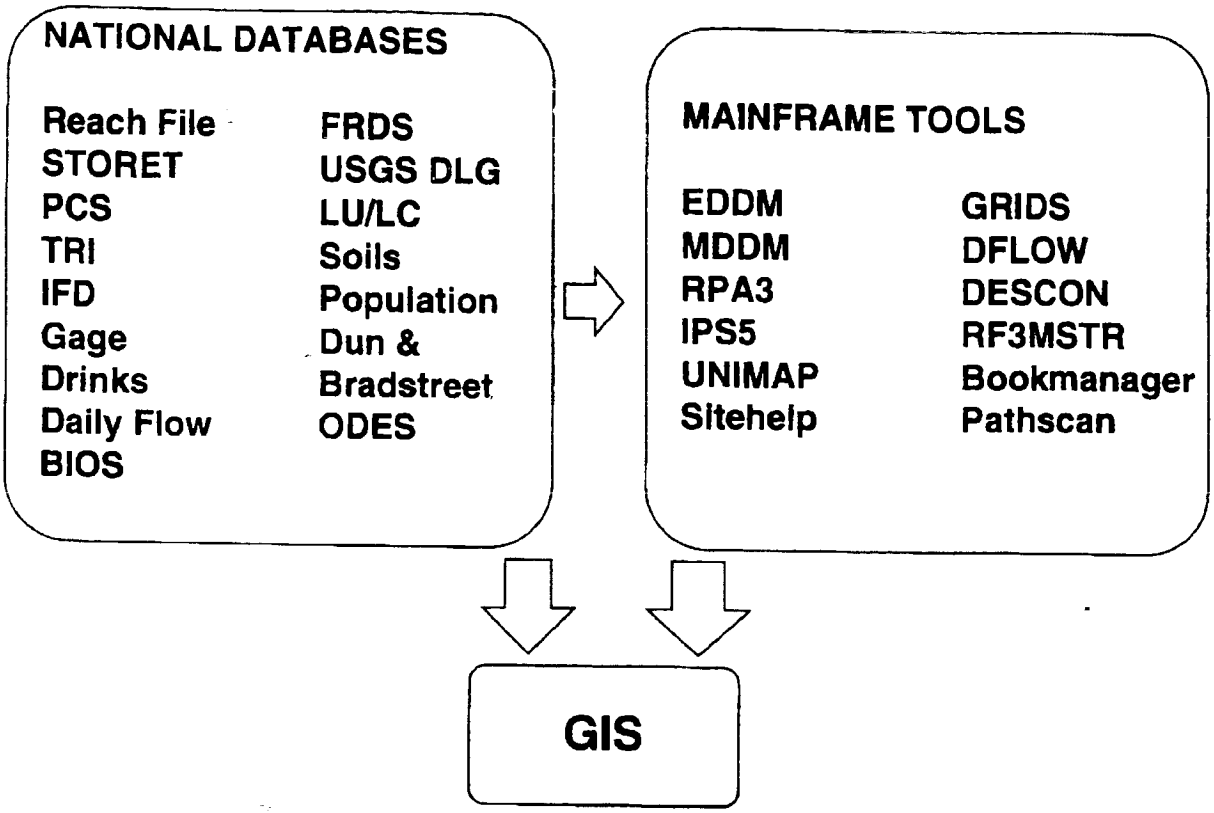

Figure 1. EPA mainframe databases/tools and linkage to GIS.

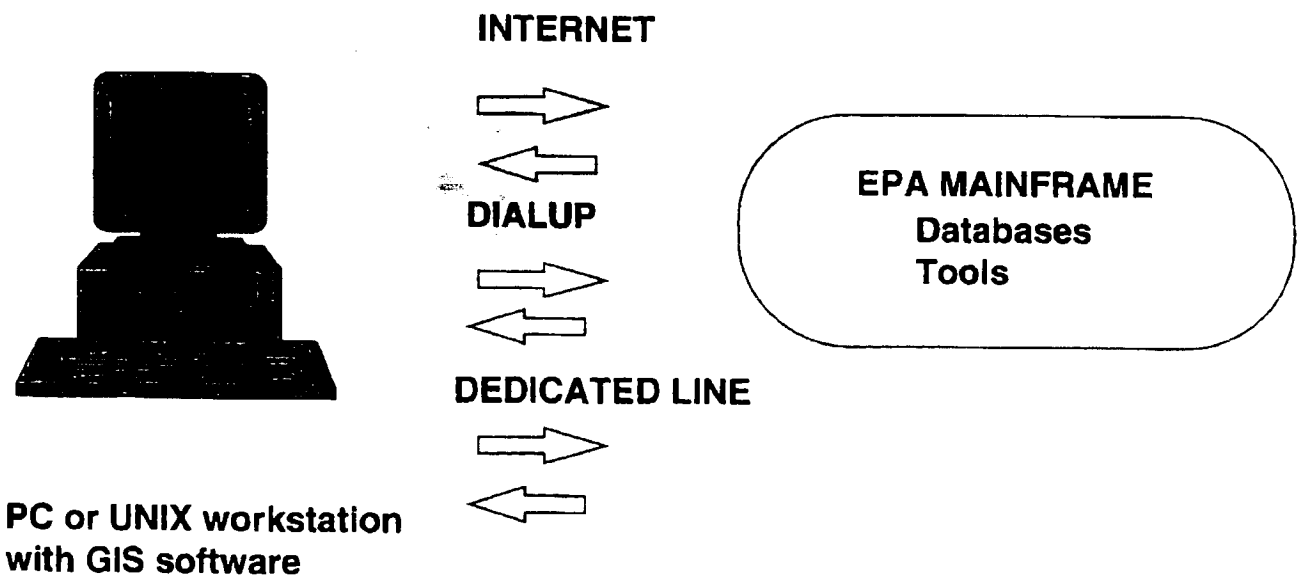

Figure 2. Access to the EPA mainframe databases and tools.

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LOCAL WORKSTATION

HARDWARE

UNIX workstation (with X-Windows) on Internet (i.e., DG Aviion)

SOFTWARE

X3270 Software - creates an X-Window, which emulates an IBM 3270 full-screen session (provided by EPA)

EPA MAINFRAME ACCOUNT

GDDMXD software provided on the mainframe to map GDDM graphics into X-Windows

Figure 3. Hardware and software requirements for accessing water quality data integration tools via INTERNET.

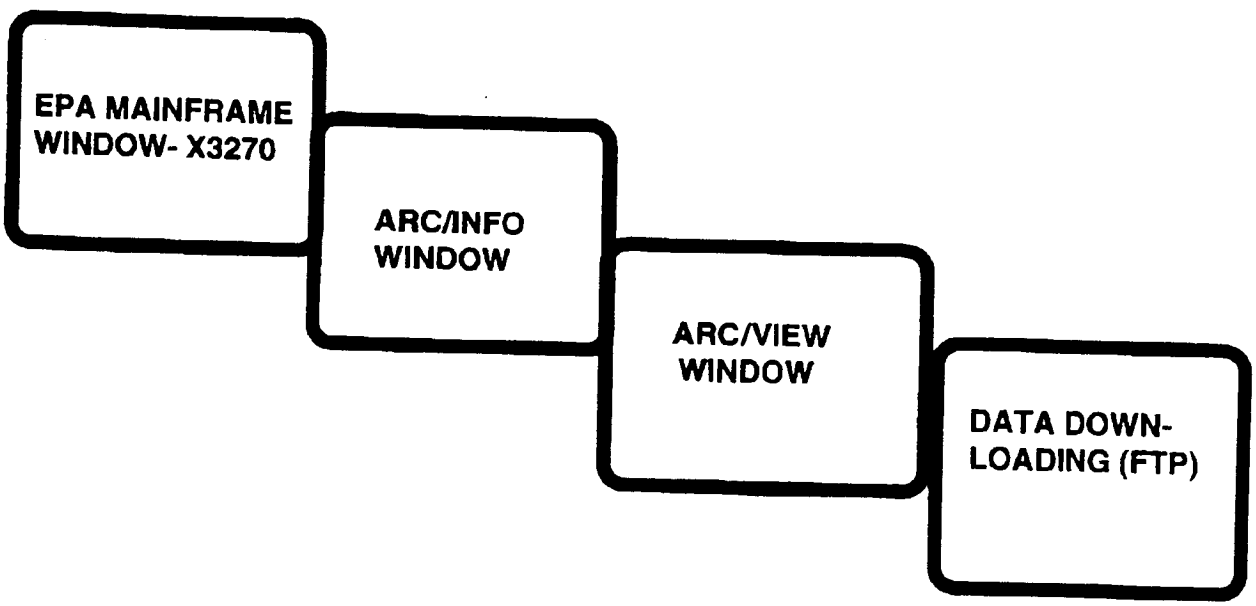

Figure 4. A UNIX workstation environment provides access to mainframe and workstation tools and databases on one desktop.

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limited local, state, and federal financial and human resources (6).

This application illustrates how the watershed approach used GIS and EPA mainframe databases and tools. As an example, a four-step approach (see Figure 5) has been developed and applied to an impaired watershed (Saluda River basin) in South Carolina. Steps one and two identified watersheds of concern through their nonattainment of designated uses (see Figure 6) and highlighted the cause of nonattainment, in this case pathogens (see Figure 7). The data sets used were U.S. Geological Survey (USGS) hydrologic unit boundaries, Soil Conservation Service (SCS) watershed boundaries, and data from the EPA waterbody system, which were indexed to the SCS watersheds.[1] These data sets were integrated into an ARC/INFO arc macro language (AML) to allow users to pose queries and prioritize watersheds for further investigation.

Once priorities were set, the third step was to evaluate, in detail, the sources and causes of nonattainment. The Saluda River basin, which had pathogens as its cause of nonattainment, was selected for further analyses. In this step, the mainframe tools supplement the workstation GIS capabilities illustrated so far. A STORET retrieval was performed for ambient water quality stations monitoring for fecal coliforms. The STORET stations were partitioned into three categories (low, medium, and high) according to the state fecal coliform standard (7), which reads:

> not to exceed a geometric mean of 200/100 mL, based on five consecutive samples during any 30 day period; nor shall more than 10% of the total samples during any 30 day period exceed 400/100 mL

The categories in Figure 8 correspond to the standard as follows:

low: < 200/100 milliliters

200/100 milliliters ≤ medium < 400/100 milliliters

high: ≥ 400/100 milliliters

Figure 8 illustrates the use of ARC/VIEW to visualize the location of fecal coliform "hot spots" in the Saluda River basin. Figure 9 focuses on one SCS watershed (03050109-040) where pathogens cause nonattainment. The locations of industrial and municipal dischargers are plotted, and facilities with fecal coliform limits and their respective permit expiration dates (captured from the PCS) are shown in a table included with the figure. The GIS capabilities used to generate Figure 9

[1] Clifford, J. 1994. Personal communication with Jack Clifford, U.S. EPA, Washington, DC.

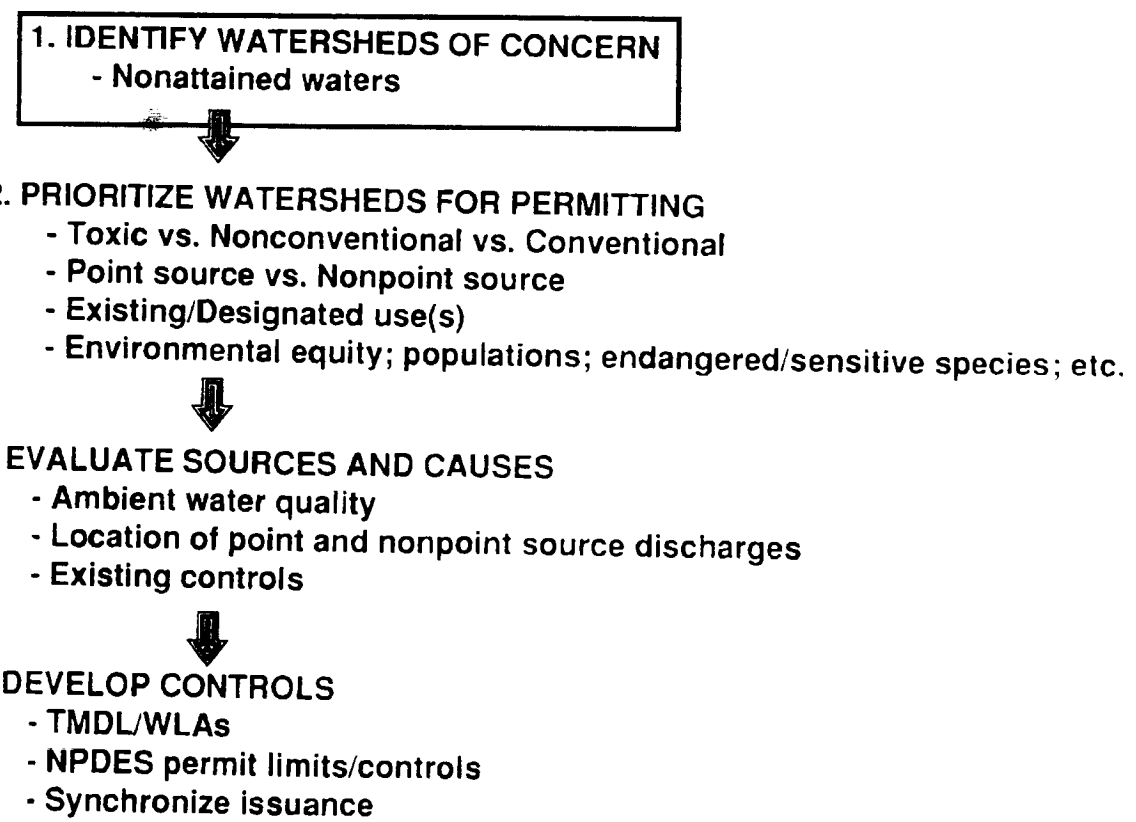

Figure 5. Four steps illustrating an example approach to permitting on a watershed basis.

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Step 1: Identify Watersheds of Concern

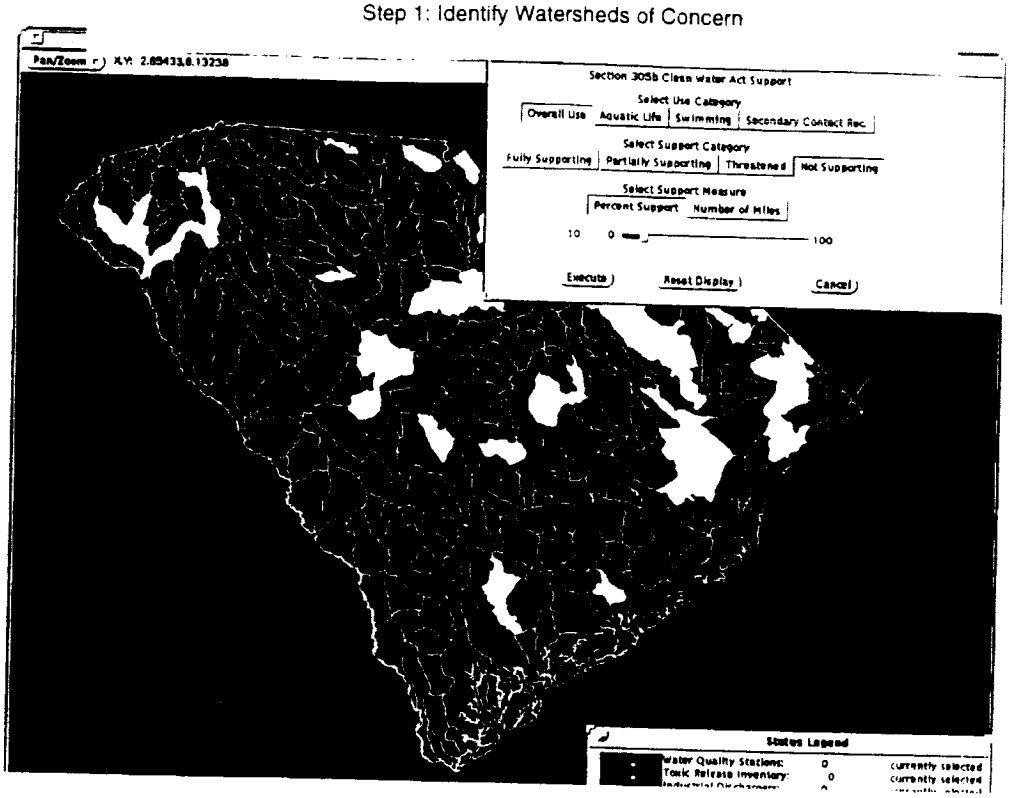

Figure 6. Identification of watersheds of concern—watersheds where at least 10 percent of the reaches are not fully supporting overall designated use (data and ARC/INFO AMLs provided by Jack Clifford).[2]

Step 2: Evaluate Priorities

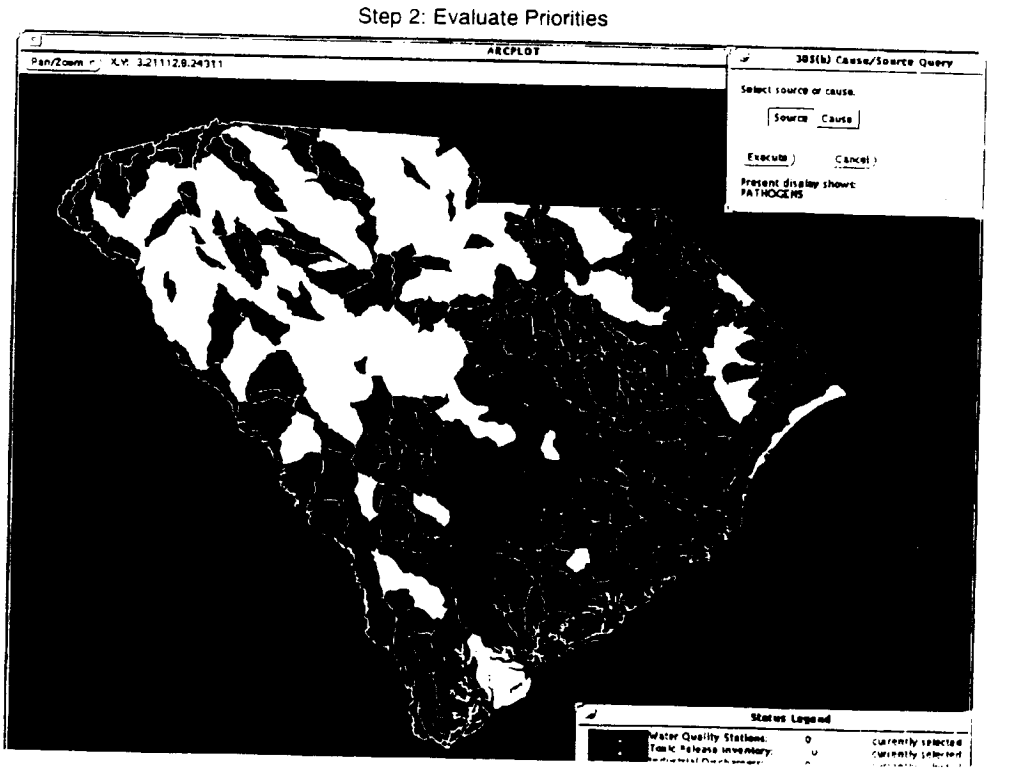

Figure 7. Watersheds where the cause of nonattainment is pathogens (data and ARC/INFO AMLs provided by Jack Clifford).[3]

[2] See note 1.
[3] See note 1.

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Step 3: Evaluate Ambient Water Quality—Low, Medium, and High Levels of Fecal Coliforms

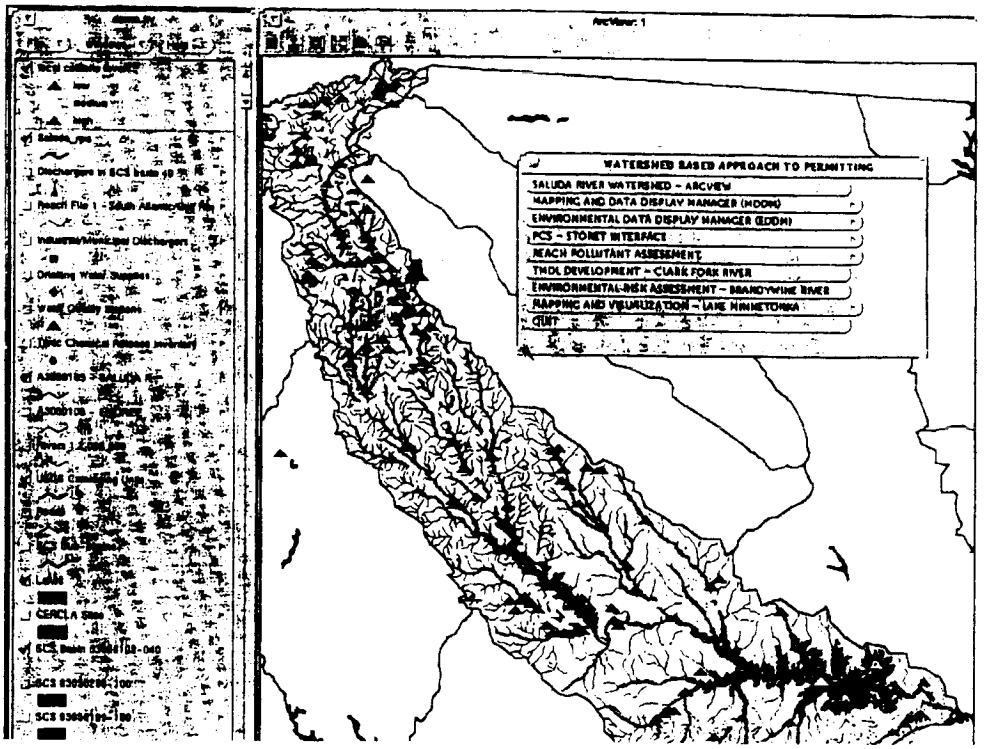

Figure 8. Map of the Saluda River basin showing the location of STORET monitoring stations and fecal coliform levels.

Step 3: Existing Controls—Dischargers That Have Limits for Fecal Coliforms

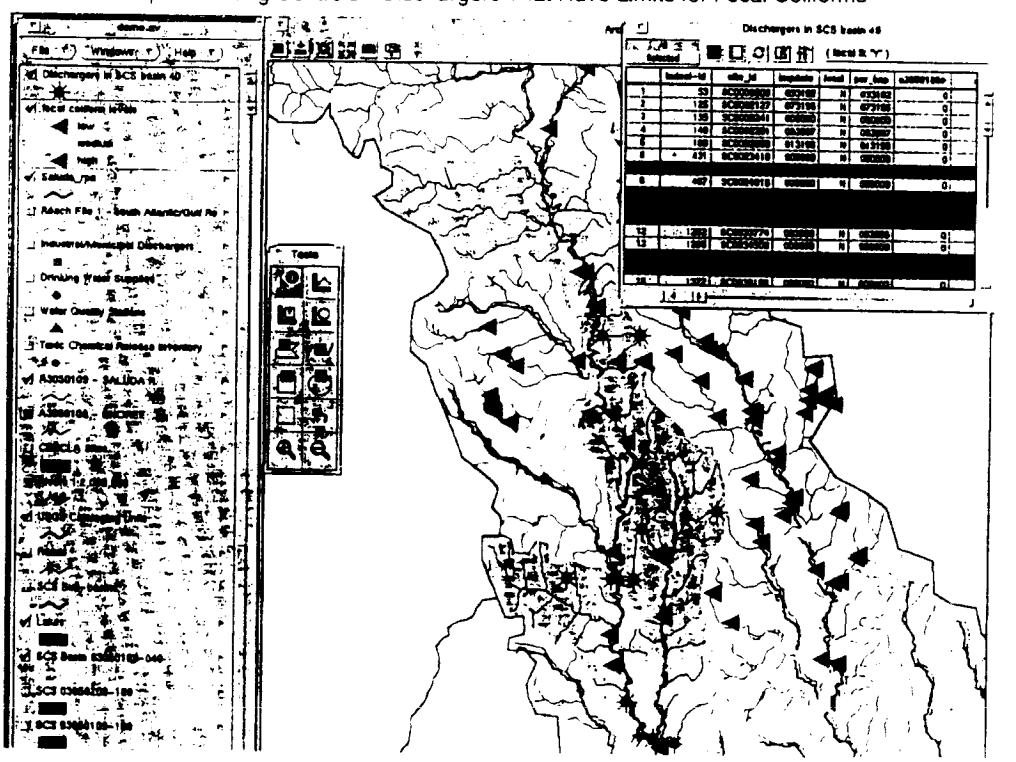

Figure 9. Focus on SCS watershed 03050109-040 (shaded in gray). Locations of STORET stations, municipal and industrial dischargers are also shown. The table in the upper right highlights dischargers with fecal coliform limits and their respective permit expiration dates.

show that permit issuance is not synchronous, which is a key element in the watershed approach.

Within this local workstation GIS environment, the attributes associated with the STORET monitoring stations and the PCS dischargers are limited with respect to the large amount of time series sampling data that exists in these databases. Figures 10 through 12 illustrate how an EPA mainframe procedure, the Environmental Data Display Manager (EDDM), can be accessed from an X-Window on the GIS workstation to query the entire STORET and PCS databases and thus provide additional data analysis and display capabilities to the GIS workstation. In Figures 10 and 11, a water quality inventory was performed for a STORET station, and a time series plot of fecal coliform levels is displayed. In Figure 12, the limits and discharge monitoring report (DMR) data were accessed from PCS for a sewage treatment plant. Excursions beyond the PCS limits for fecal coliforms are easily visualized in the plot.

The fourth and final step in the watershed approach was to develop controls for achieving water quality standards. This might include the development of total maximum daily loads (TMDL), waste load allocations (WLA), and the synchronization of permit issuance. Another mainframe tool, the PCS-STORET INTERFACE (referred to as IPS5 on the mainframe), can be used to access and compute design flows for TMDL development (see Figure 13) and to find all facilities discharging to a particular reach (see Figure 14), an initial step in the synchronization of permit issuance.

RPA

The RPA is a procedure on the EPA mainframe that automates identification of reaches where priority pollutants have been detected. It can be run for a user-selected state or USGS hydrologic unit.

Section 304(l) of the Clean Water Act (CWA) identifies water bodies impaired by the presence of toxic substances,

Step 3: Using EDDM To Evaluate Ambient Upstream Fecal Coliform Levels—Regulation of Upstream Dischargers Is Necessary

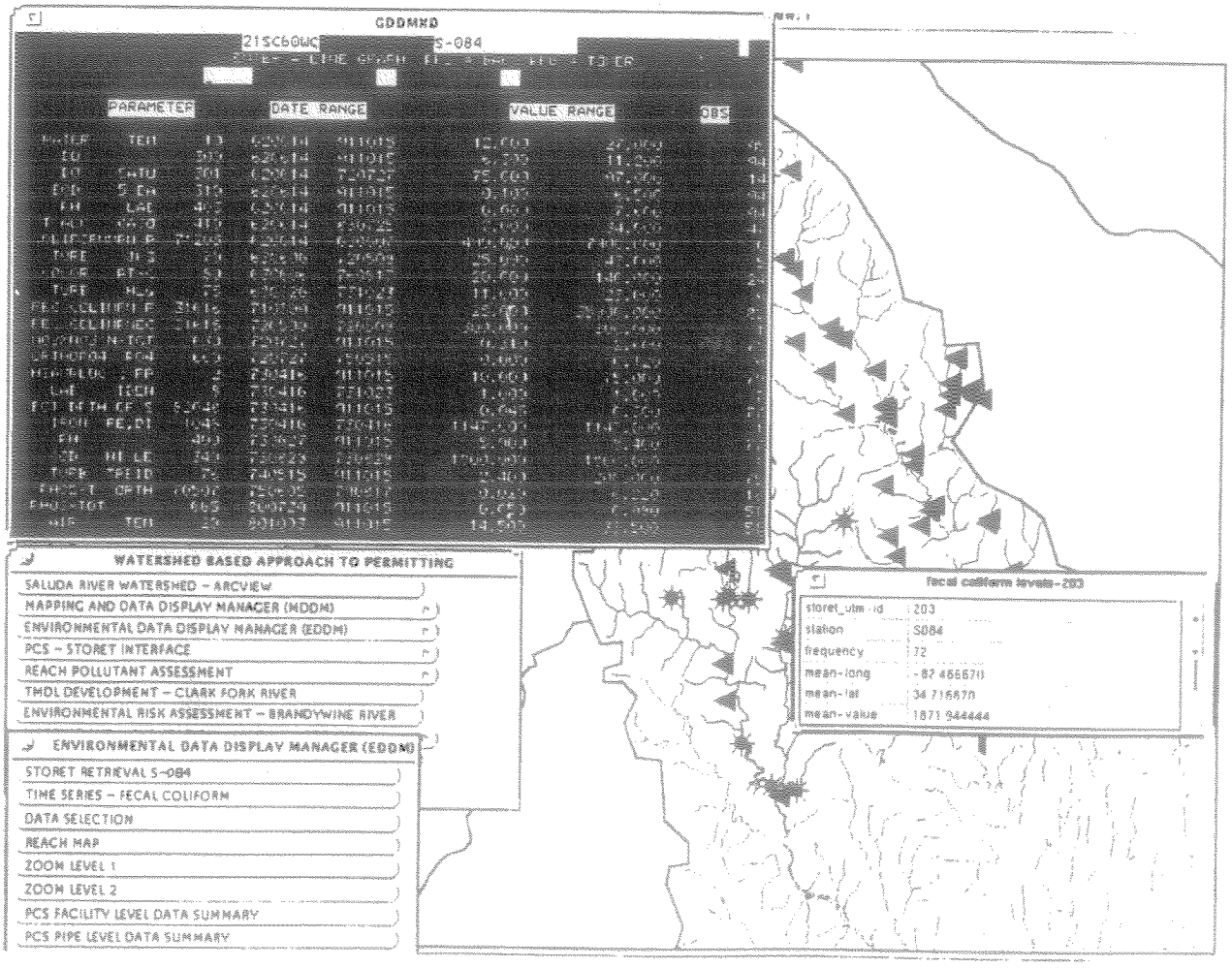

Figure 10. Using EDDM to perform a water quality inventory for STORET station 21SC06WQ S-084 in the Saluda River basin, South Carolina.

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Step 3: Using EDDM To Visualize Trends in Ambient Coliform Levels Upstream of a Discharger

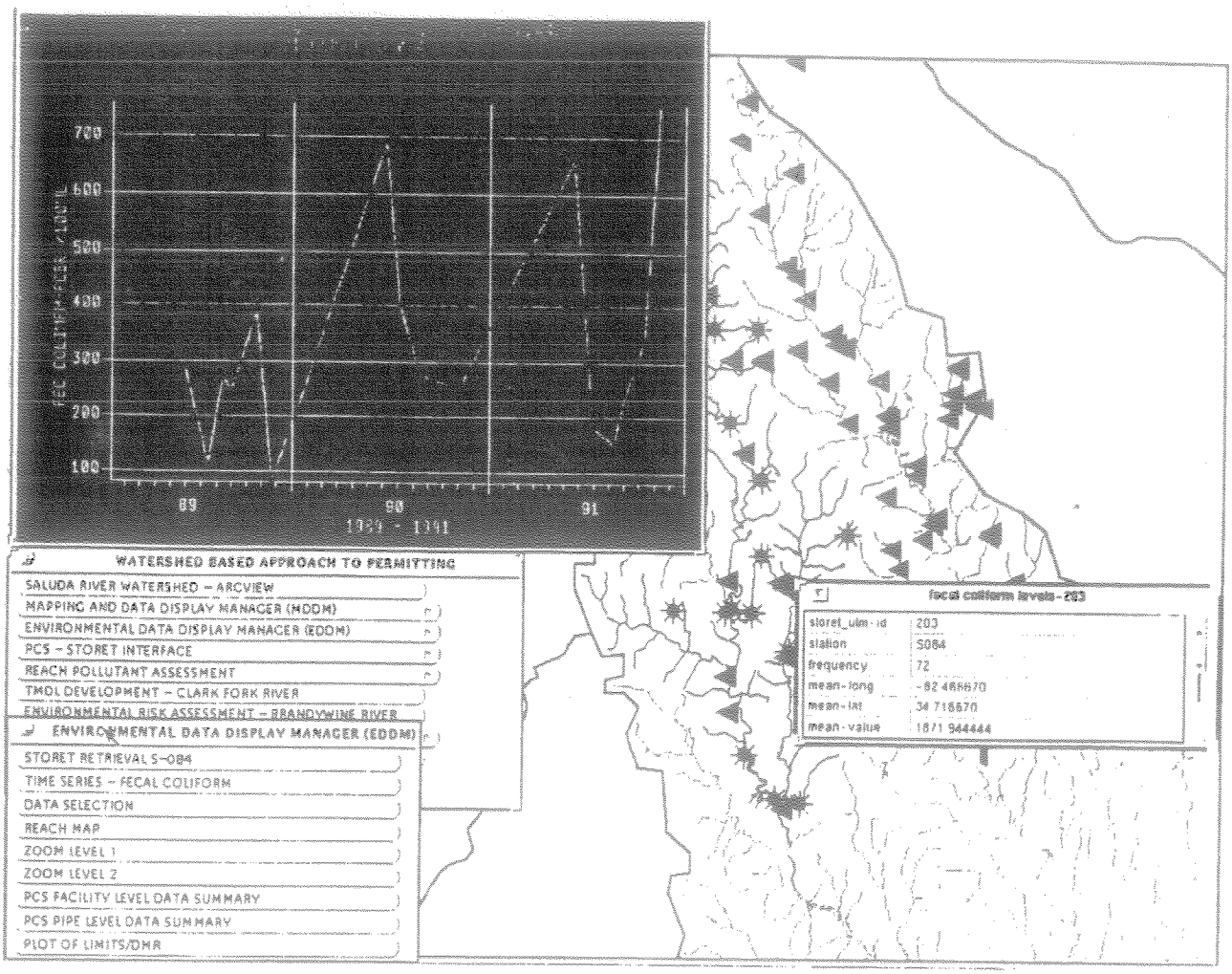

Figure 11. From the EDDM water quality inventory table, the fecal coliform parameter (31616) was selected for a time series plot.

identifying point source dischargers of these substances and developing individual control strategies for these dischargers. To meet these requirements, the EPA Office of Wetlands, Oceans, and Watersheds prepared guidance identifying criteria to be used in reviewing state reports.

The RPA was designed to address the requirements under criterion 7 of Section 304(l): identification of state waters with likely presence of priority toxic pollutants. This assessment was accomplished by identifying and summarizing reaches with point source dischargers of priority pollutants and water quality stations with priority pollutant data.

Information on the state's waters is summarized using the USGS hydrologic unit naming convention and the Reach Structure File (Version 1). Numerous databases were accessed and analyzed, including the Reach Structure and Reach Trace File (Version 1), industrial facilities discharge (IFD) file, STORET parameter file, PCS, and the STORET water quality file. The IFD file and PCS provided the facility information. Comparing information from both data sources identified active facilities and generated a complete list of facilities by their assigned reach numbers. Water quality data from STORET were summarized on reaches with priority pollutant monitoring data. Stations were retrieved with the following restrictions:

- Stations located within the state or hydrologic unit of interest.
- Ambient monitoring stations located on streams, lakes, or estuaries.
- Stations sampled for at least one priority pollutant in either water, sediment, or fish tissue on or after January 1, 1982.

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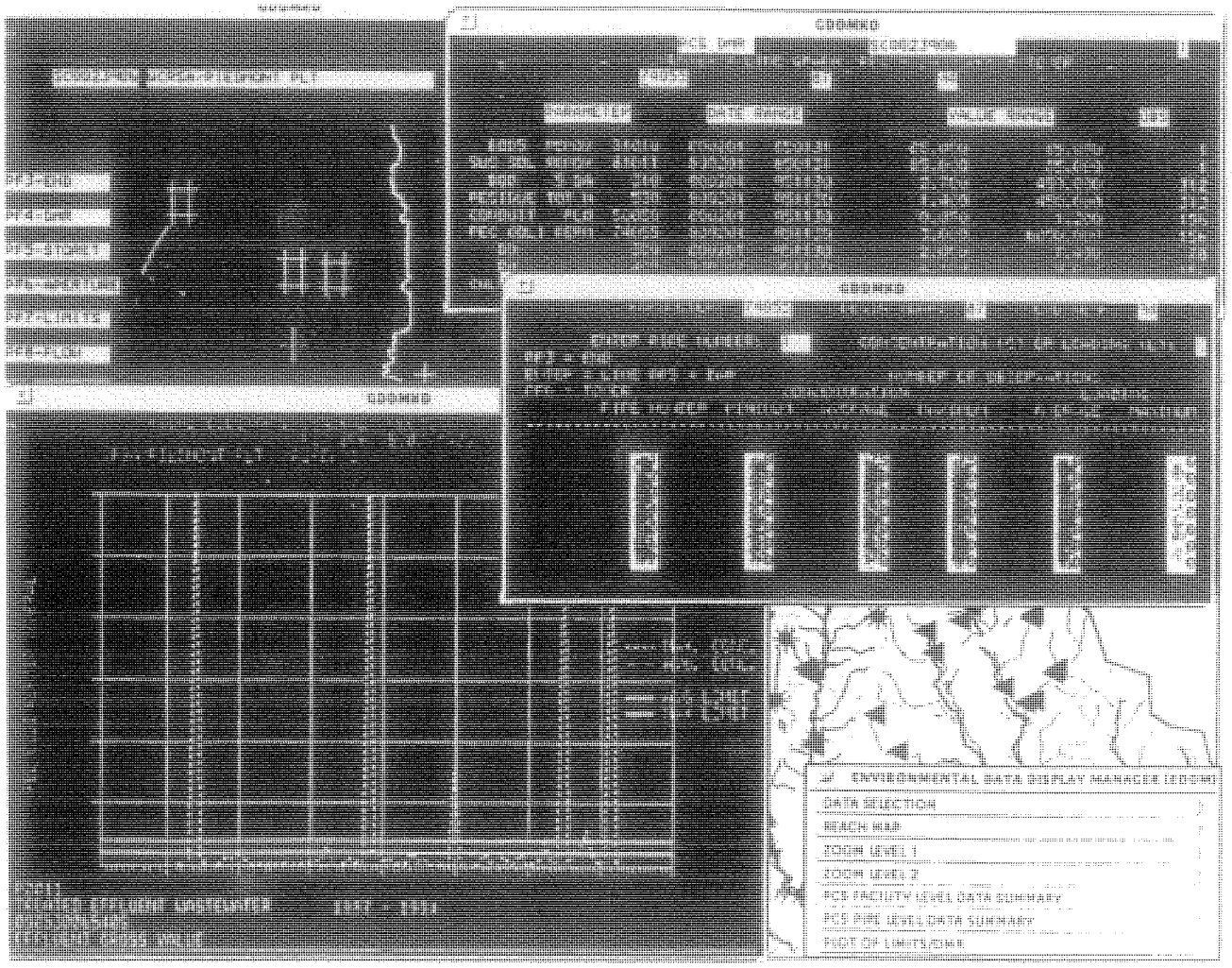

Figure 12. Using EDDM, PCS data is accessed for discharger SC0023906, Piedmont sewage treatment plant. The windows show plant location (upper left), facility and pipe summary data (upper right) and time series plot (lower left).

Recently, the RPA program was modified to output files compatible with GIS. An example of this is shown in Figure 15. The data in the table portion of this figure are written to two files as follows:

- For each reach in the hydrologic unit, the geographic coordinates are written to a file in ARC/INFO GENERATE format.
- The attributes associated with each reach (e.g., name, length, number of water quality stations) are written to a delimited ASCII file.

A third file is also automatically generated. This file is an AML that GENERATES the line coverage of reaches, defines and populates the INFO table of attributes, then joins the attributes in the INFO table to the line coverage of reaches. Once these three files are created, they are downloaded to the GIS workstation (via ftp) and processed by ARC/INFO. In ARC/VIEW, the user can identify a reach and determine:

- The number of water quality stations with priority pollutant monitoring data.
- The number and type of industrial facilities with priority pollutant discharge.
- The number of publicly owned treatment works (POTW) with and without indirect dischargers (see Figure 15).

In addition to this reach summary data, other tables (cross-linked to reaches, water quality stations, and dischargers) are produced that summarize the data by pollutant (see Figure 16). In this figure, each pollutant is cross-linked to the reach where it was detected, and the source of detection is also identified (i.e., water column, sediment, fish tissue, NPDES permit limit, Form 2(c) submittal) or

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Step 4: Using the PCS/STORET INTERFACE for NPDES Permit Development—Analysis of Receiving Stream Flow Data

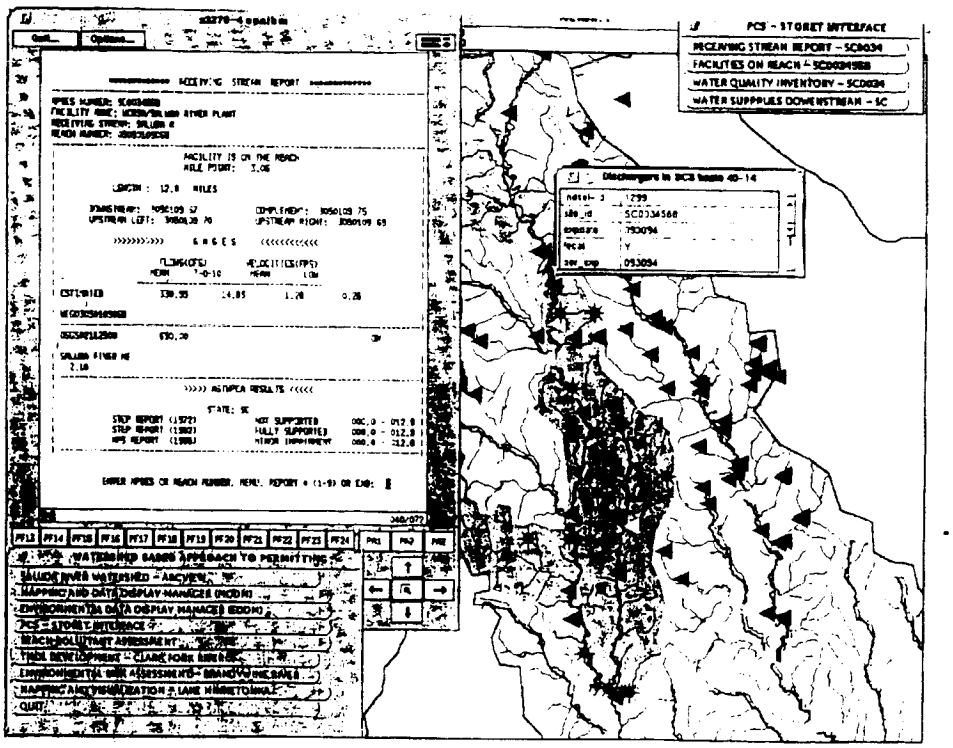

Figure 13. Using the PCS-STORET INTERFACE to access and compute design flows for a specific reach.

Step 4: Using the PCS/STORET INTERFACE To Determine Other Facilities on a Reach for Wasteload Allocation Purposes

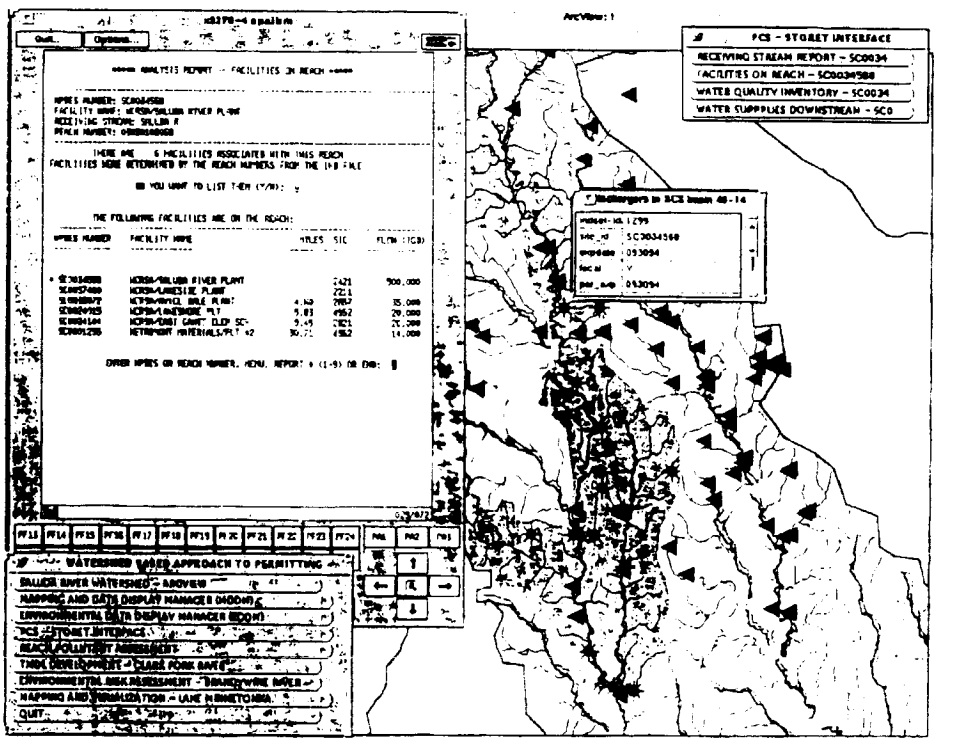

Figure 14. Using the PCS-STORET INTERFACE to list all facilities that discharge to a specific reach.

predicted to be in the discharger's effluent based on the standard industrial classification (SIC) code. More detailed information is also generated. For example, Figure 17 shows a detailed report for priority pollutants detected in the water column (similar reports are generated for sediment and fish tissue). Each pollutant is cross-linked to a reach and the specific monitoring station where it was detected. Basic summary statistics are also presented.

Figure 18 shows a detailed report for pollutants detected in the NPDES permit limit. In this figure, each pollutant is cross-linked to a reach and the specific NPDES discharger containing a permit limit. In addition, each discharger is identified as a major, minor, or POTW.

Figures 15 through 17 show the RPA output in the foreground and coverages displayed by ARC/VIEW in the background. Inspection of these figures shows that the Bush River is a priority pollutant reach containing seven industrial facilities, four of which discharge priority pollutants (one pulp and paper mill, three textile factories). Further examination of the data shows that cadmium has been detected in the water column and is also contained in the NPDES permit limit. It is also predicted to be in the discharge effluent based on SIC code. In the water column, cadmium was measured at 10 µg/L at two stations sampled in 1988.

Finally, there is a limit for cadmium in NPDES facility SC0024490 (Newberry plant), a POTW on the Bush River. The RPA output, linked to GIS, can be used as a screening and targeting tool for identifying specific reaches within watersheds where toxic priority pollutants cause water quality degradation.

Summary and Conclusions

The proliferation of GIS workstations, the expansion of the Internet, and the development of X-Window-based graphics emulation software (X3270 and GDDMXD) has afforded analysts the opportunity to use the powerful analytical capabilities of GIS and the EPA mainframe databases and tools together on one desktop. Thus, a user performing a GIS watershed analysis can also have immediate and complete access to national on-line databases such as STORET and PCS by opening up a "window" to the EPA mainframe. This allows detailed queries to be performed that supplement the data already being analyzed at the local workstation. This capability allows users to easily visualize additional data without having to spend effort in retrieval, downloading, transforming, and reformatting to make it useful. By enhancing existing mainframe programs to create output in GIS format, the time spent importing data to the GIS is reduced and more time can be spent on analysis. An example of this capability is the RPA program.

Reach Pollutant Assessment: Priority Pollutant Detection in the Bush River

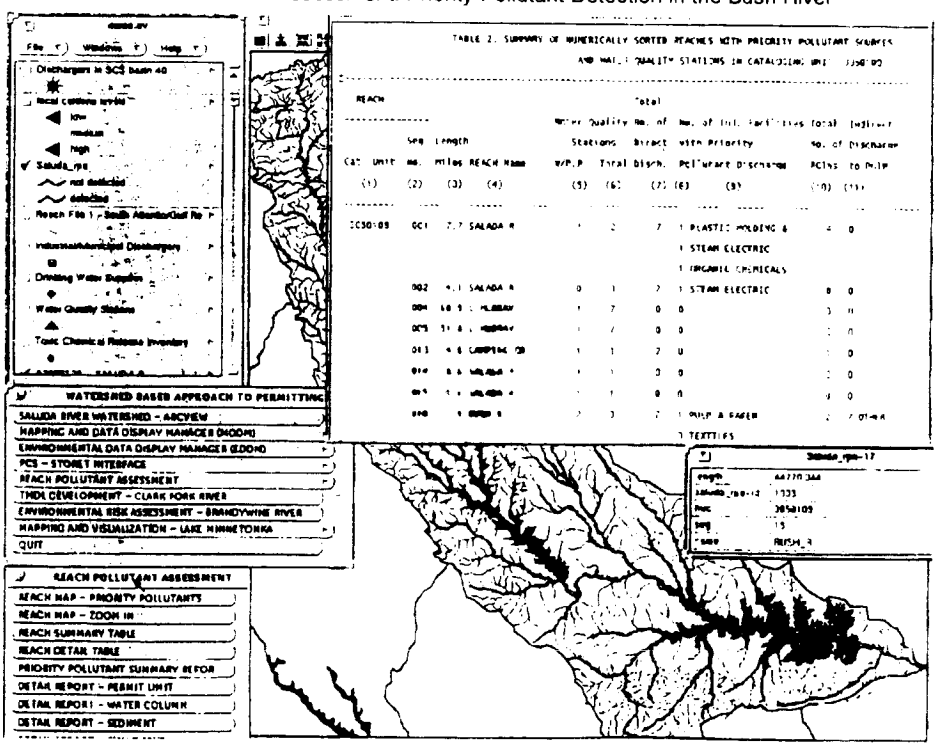

Figure 15. Using the RPA procedure to identify specific reaches with priority pollutants.

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Reach Pollutant Assessment: Cadmium in the Bush River

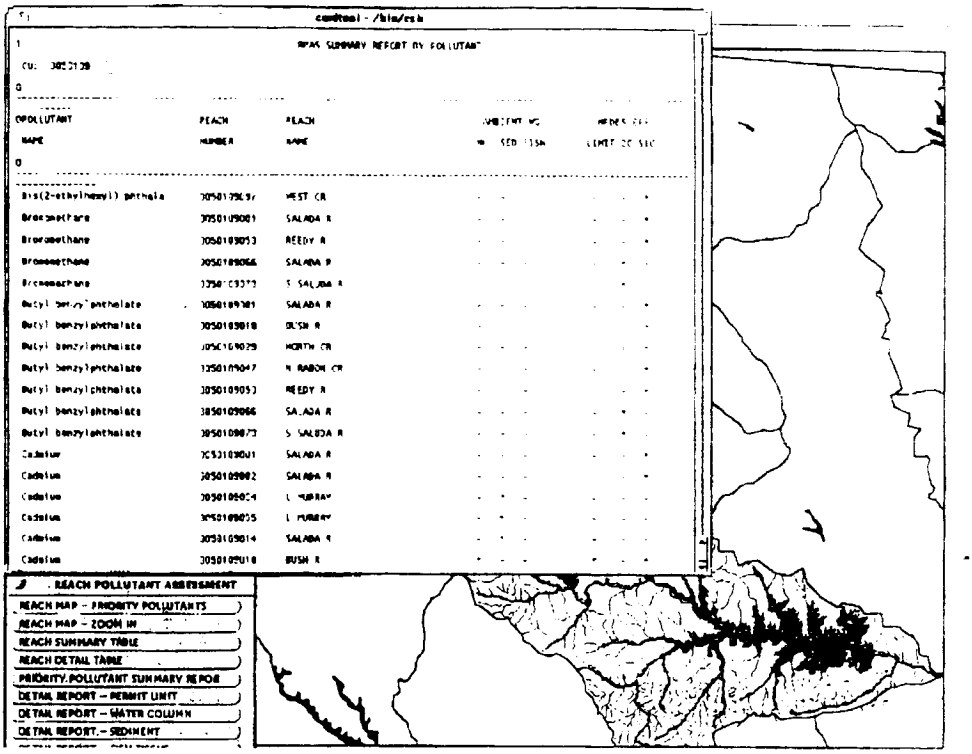

Figure 16. RPA summary report by pollutant.

Reach Pollutant Assessment: Bush River: Cadmium in the Water Column

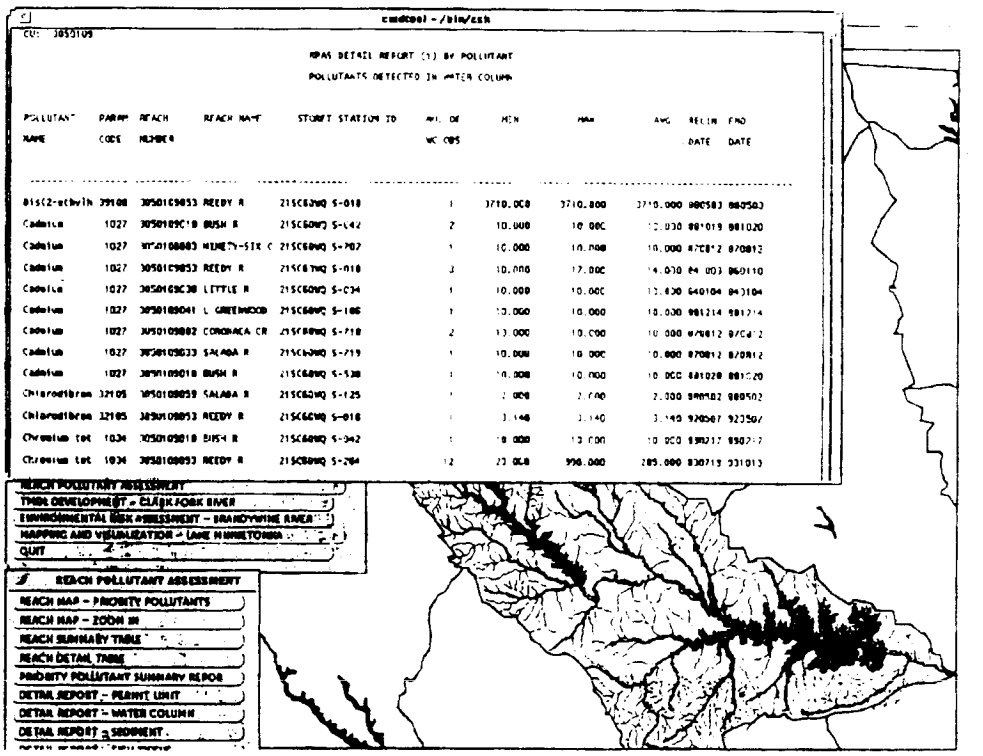

Figure 17. RPA detail report: pollutants detected in the water column.

Reach Pollutant Assessment: Permitted Industries for Cadmium—Bush River

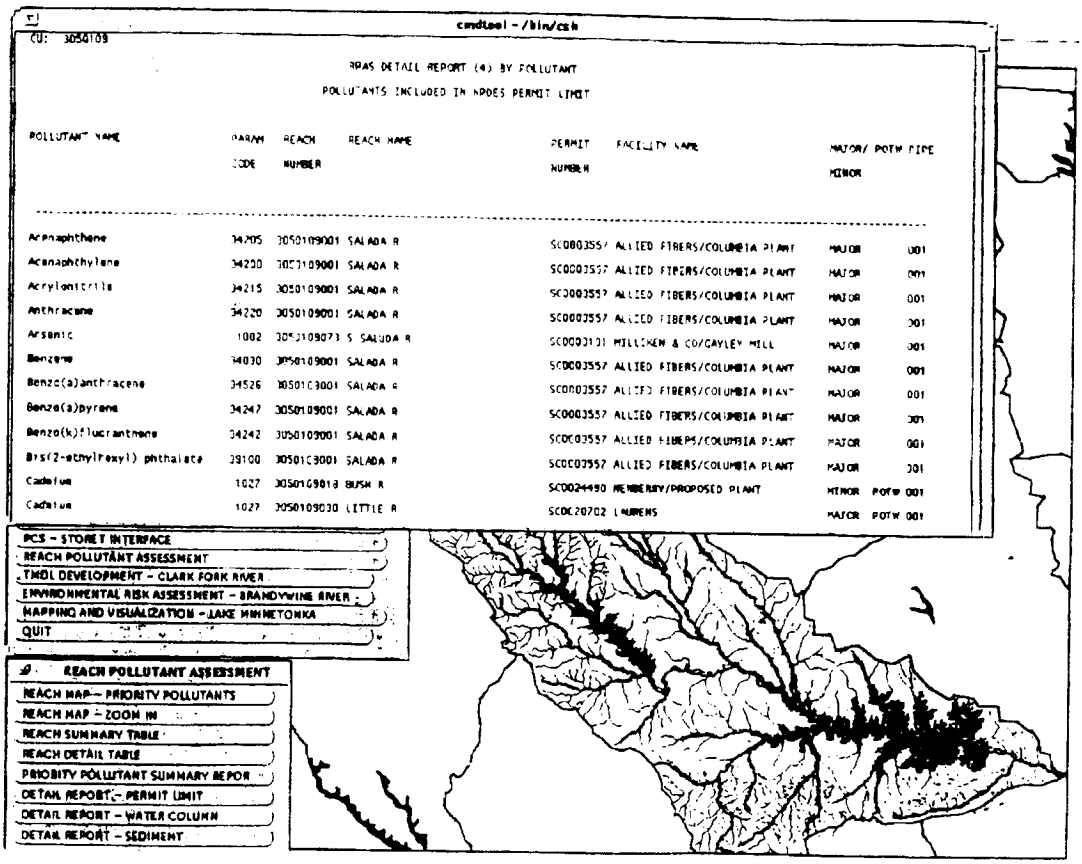

Figure 18. RPA detail report: pollutants included in the NPDES permit limit.

Two examples were presented as illustrations of how GIS and the mainframe databases and tools can work together.

In the first example, EPA's watershed-based approach to permitting, a four-step approach, was outlined, showing how a combination of local GIS functions and remote mainframe databases and tools were used in each step of the process. The end result was the targeting and prioritizing of watersheds of concern, and a detailed look at where and why water quality standards were not being met.

In the second example, the RPA program along with GIS was used to identify and map toxic priority pollutants and cross-link them to reaches, media (water column, sediment, fish tissue), NPDES dischargers, and monitoring stations. This analysis focused on what the toxic pollutant problems are and where they occur.

Acknowledgments

The authors wish to thank Jack Clifford, AWPD, OWOW, EPA, for providing data and ARC/INFO AMLs that were used in the South Carolina watershed example. The authors also extend their appreciation to Duane More and Tom Lewis (both with Martin Marietta), who provided support in setting up the X3270 and GDDMXD software.

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Wetlands Applications

Wetlands Mapping and Assessment in Coastal North Carolina: A GIS-Based Approach

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Introduction

The coastal area of North Carolina covers 20 counties and over 9,000 square miles of land area, about 20 percent of the state (see Figure 1). It also includes over 87 percent of the state's surface water. The North Carolina Coastal Management Program (NC CMP) is responsible for managing this area to meet the goals set forth in the Coastal Area Management Act (CAMA) (North Carolina General Statute [NCGS] 113A, Article 7). These goals provide a broad mandate to protect the overall environmental quality of the coastal area and to guide growth and development in a manner "consistent with the capability of the land and water for development, use, or preservation based on ecological considerations" (NCGS 113A-102(b)(2)).

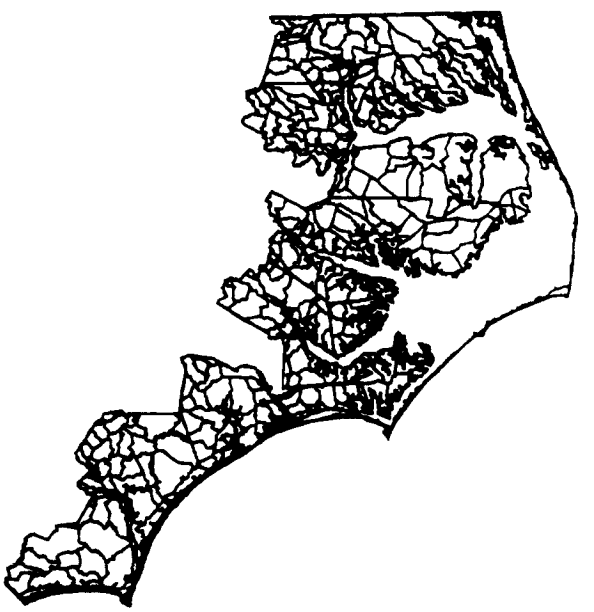

Figure 1. HU and county boundaries in the North Carolina coastal area.

Much of the North Carolina coastal area consists of wetlands, which, in many areas, constitute nearly 50 percent of the landscape. These wetlands are of great ecological importance, in part because they occupy so much of the area and are significant components of virtually all coastal ecosystems, and in part because of their relationships to coastal water quality, estuarine productivity, wildlife habitat, and the overall character of the coastal area.

Historically, close to 50 percent of the original wetlands of the coastal area have been drained and converted to other land uses (1-3). Although agricultural conversion, the largest historical contributor to wetlands loss, has largely stopped, wetlands continue to disappear as they are drained or filled for development. Conflicts between economic development and wetlands protection continue to be a major concern, with many coastal communities considering wetlands protection to be a major barrier to needed economic development.

Because wetlands are such a dominant part of the coastal landscape and are vitally important to many aspects of the area's ecology, their management and protection is a major concern of the NC CMP. The State Dredge and Fill Act (NCGS 113-229) and the CAMA regulatory program stringently protect tidal wetlands, or "coastal wetlands" as law and administrative rules call them. Coastal wetlands are designated areas of environmental concern (AECs), with the management objective "to give highest priority to the protection and management of coastal wetlands so as to safeguard and perpetuate their biological, social, economic and aesthetic values; and to coordinate and establish a management system capable of conserving and utilizing coastal wetlands as a natural resource essential to the functioning of the entire estuarine system" (15A NCAC 7H .0205).

North Carolina law does not, however, specifically protect nontidal freshwater wetlands. State protection of

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freshwater wetlands is limited to the regulatory authority provided under federal laws for state agency review of federal permits; in this case, §404 permits granted by the U.S. Army Corps of Engineers. Under §401 of the Federal Water Pollution Control Act (33 USC 1341), a Water Quality Certification from the North Carolina Division of Environmental Management (DEM) is required for a 404 permit to discharge fill material into wetlands. Section 307 of the federal Coastal Zone Management Act (CZMA - 16 USC 1451 et seg.) also requires that 404 permits be consistent with the enforceable rules and policies of the NC CMP. The standards for consistency are the use standards for AECs and wetlands policies stated in the applicable local land use plan. Other than AECs, the NC CMP has no consistent policies regarding wetlands. A few local land use plans include policies to protect freshwater wetlands, but most do not.

Wetlands Conservation Plan

In 1991, the CZMA §309 Assessment of the NC CMP revealed NC CMP's weakness in protecting nontidal wetlands (4). The assessment demonstrated that both opponents and proponents of wetlands protection considered the current system inadequate. Economic development interests found the 404 regulatory program to be unpredictable and inconsistent, often resulting in the loss of needed economic growth in coastal counties. Environmental interests felt that the program allowed the continued loss of ecologically important wetlands. As a result, the assessment identified wetlands management and protection as one of the primary program areas in need of enhancement.

The North Carolina Division of Coastal Management (DCM) developed a 5-year strategy (5) for improving wetlands protection and management in the coastal area using funds provided under the Coastal Zone Enhancement Grants Program established by 1990 amendments to §309 of the federal CZMA. The Office of Ocean and Coastal Resources Management (OCRM) in the National Oceanographic and Atmospheric Administration (NOAA), U.S. Department of Commerce administers the §309 program. Funds provided under this program, particularly Project of Special Merit awards for fiscal years 1992 and 1993, supported the work reported in this paper. A grant from the U.S. Environmental Protection Agency (EPA) for a Wetlands Advance Identification (ADID) project in Carteret County, North Carolina, also funded this work.

The key element of DCM's strategy for improving wetlands protection is the development of a wetlands conservation plan for the North Carolina coastal area. The plan has several components:

- Wetlands mapping inventory
- Functional assessment of wetlands
- Wetlands restoration
- Coordination with wetlands regulatory agencies
- Coastal area wetlands policies
- Local land use planning

The obvious first step in developing a wetlands conservation plan is to describe the wetlands resource. An extensive geographic information system (GIS) wetlands mapping program is helping to accomplish this first step by producing a GIS coverage of wetlands by wetland type for the entire coastal area. The GIS coverage allows generation of paper maps for areas within any boundaries available in GIS format. This is the subject of the first part of this report.

One weakness of the 404 program is that, for individual permits, it attempts to apply the same rules and procedures equally to all wetlands, regardless of the wetland type and location in the landscape. This approach can result in permits being granted for fill of wetlands of high ecological significance or permits being denied to protect wetlands of little significance. Neither outcome is desirable because the result may be the loss of either vital wetland functions or beneficial economic activity. This is an unsatisfactory way to manage wetland resources in an area such as the North Carolina coast, where:

- A high proportion of the land is wetlands.
- Many of the wetlands are vital to the area's environmental quality.
- Economic stimulation is sorely needed.

To help overcome this weakness in the current wetland regulatory framework, the Wetlands Conservation Plan includes an assessment of the ecological significance of all wetlands to determine which are the most important in maintaining the environmental integrity of the area. This will result in a designation of each wetland polygon in the GIS coverage as being of high, medium, or low functional significance in the watershed in which it exists. The procedure by which this occurs is the subject of the second part of this report.

The remaining components of the Wetlands Conservation Plan comprise the means by which the results of the wetland mapping and functional assessment steps will be used to improve wetland protection and management. Close coordination with other state and federal agencies involved in wetlands protection and management has been an important component of the entire effort. Agency representatives have been involved in developing the methods used, and the agencies will receive copies of the resulting maps for use in their own planning and decision-making. Policies for protection of wetlands of varying functional significance will be proposed to the Coastal Resources Commission to serve as the

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basis for consistency review of 404 permit applications. Wetland maps and functional assessment results will also be provided to local governments for use in local land use planning, and DCM will work with local governments to increase local involvement in the wetlands regulatory structure.

While the wetland maps themselves are useful for land use planning and helping to find suitable development sites, simply knowing where the wetlands are located is insufficient information for many purposes. Any area for which a 404 permit application is in process has been officially delineated as a wetland by the Corps of Engineers. The value of wetland maps to the regulatory review agencies at this stage is limited to determining the relationship of the site to other wetlands in the area. While, ideally, all wetlands should be avoided in planning development, avoiding wetlands completely in the coastal area is difficult, and avoiding all wetlands in any extensive development is virtually impossible.

The results of the functional assessment will provide additional information about the ecological significance of wetlands. This information will be valuable to wetland regulatory review agencies in determining the importance to an area's environmental integrity of protecting a particular site for which a permit to fill has been requested. It will also enable development projects to be planned so as to avoid, at all reasonable costs, the most ecologically important wetlands. An accurate functional assessment of wetland significance, then, is the most valuable component of the Wetlands Conservation Plan.

Wetlands Mapping Inventory

An important, initial step in developing a comprehensive plan for wetlands protection is to understand the extent and location of wetlands in the coastal area. When developing mapping methods, DCM quickly realized that the more than 9,000-square-mile coastal area was too large for any mapping effort in the field (see Figure 1). To complete this task in an accelerated timeframe, DCM needed to use existing data compatible with GIS. Reviewing the existing data revealed that most are not applicable for one of two reasons: (1) available wetlands data are based on older photography, and (2) more recent data are not classified with the intent of wetlands mapping. These data types, used independently, are inappropriate for use in a coastal area wetlands conservation plan. In addition, the classification schemes used in the existing methods are too complex or not focused on wetlands.

While several data sets were believed to be inappropriate if used exclusively for wetlands mapping in coastal North Carolina, each contained useful components. DCM elected to combine three primary layers of data and extract the most pertinent information from each layer. DCM selected the National Wetlands Inventory (NWI) because its primary purpose is to map wetlands. Unfortunately, these maps were based on photography from the early 1980s in coastal North Carolina, and many changes have occurred in the landscape since that time. NWI also omitted some managed wet pine areas from its maps; DCM wished to include these areas because they are important to the ecology of the North Carolina coastal area. DCM also selected detailed soils lines for use in its mapping efforts. While soils alone should not be used to identify wetlands, soils can be very useful in identifying marginal areas. Finally, DCM also employed thematic mapper (TM) satellite imagery in its methods. This data layer was not developed as a wetlands inventory; however, the imagery is more recent than the soils and NWIs. DCM desired to incorporate the benefits of each of these data sources into its mapping techniques.

The information provided by this mapping exercise will be useful to county and municipal planners in helping guide growth away from environmentally sensitive areas. For this reason, DCM elected to pursue mapping on a county by county basis. In addition, a single county allowed DCM to focus methodology development to a limited geographic area to refine its methods. Carteret County was selected as a methods development laboratory because data were available for the area and because Carteret has a large number of representative wetlands.

Data Descriptions

The U.S. Fish & Wildlife Service produces the NWI for all wetlands in the country. For the coastal North Carolina area, these vector data were developed from 1:58,000-scale color infrared photography taken during the winters of 1981, 1982, and 1983. Photointerpreters delineated wetland polygons on clear stabilene mylar taped over the photographs. After an initial scan of the photographs to identify questions or problem signatures, the photointerpreters reviewed areas in the field. They performed approximately one-half to one full day of field verification per quadrangle (quad) (6). Features were compared with U.S. Geological Survey (USGS) topographic maps for consistency. Following completion of the 'draft' paper maps, the Regional Coordinator reviewed the data. After approval as a final map, each quad was digitized. Initially, the North Carolina Center for Geographic Information and Analysis (CGIA) digitized the coastal North Carolina NWI maps, and later, the NWI Headquarters in St. Petersburg, Florida, who subcontracted the task, digitized them. Digital maps were obtained initially from 1/4-inch tape transfer and later from direct access to NWI via the Internet.

CGIA provided digital, detailed soil lines, which also are vector data based on 1:24,000 quads. County soil scientists delineated soil boundaries on aerial photographs

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based on slope, topography, vegetative cover, and other characteristics. This process occurs in any soil survey. After appropriate personnel approved the lines, a qualified soil scientist recompiled them onto orthophoto quads. CGIA scanned or manually digitized these lines. The coverage incorporated databases describing soil characteristics, which were then released for use.

The Landsat Thematic Mapper (TM) imagery was classified as part of the Albemarle-Pamlico Estuarine Study (APES). To provide complete coverage for the southernmost region of DCM's jurisdiction (Onslow, Pender, Brunswick, and New Hanover Counties), DCM contracted with CGIA and the North Carolina State University (NCSU) Computer Graphics Center to have that area processed identically to the APES region. These data provide a raster-based coverage of approximately 30-meter pixel resolution. Some of the imagery was taken at high tide, which precludes some near-water wetlands from appearing in certain areas. Using ERDAS, imagery specialists grouped similar spectral signatures into one of 20 classes. DCM used these data in two formats: filtered and unfiltered. The unfiltered information was vectorized with the ARC/INFO GRIDPOLY command. To remove some of the background noise in the coverage, it was filtered using ERDAS 'scan' with a Majority filter of 5 by 5 pixels, then vectorized with the ARC/INFO GRIDPOLY command.

Methods

Within each county, mapping is based on 1:24,000 USGS quads. After completion, each quad is assembled into a countywide coverage, which eventually is assembled into a coastal area coverage. The initial step in the mapping process is to ensure completion of the base layers described previously. Reviewing for errors at early stages prevents confusion in correction later in the process; therefore, the importance of the preliminary techniques cannot be overemphasized. The NWI data are first inspected to ensure complete coverage. If parts of the quad are missing, the error is investigated and corrected. Omissions may be areas of severe cloud cover on the photography or areas neglected during the digitization process. Next, the coverage is reviewed for missing label points. Any omissions are corrected based on the finalized version of the published NWI paper map. Appropriate NWI staff are contacted for the necessary information. At this time, labels are verified for typographical misentry. If not corrected, these errors could lead to confusion later in the mapping process.

Once the label errors are detected and corrected, the polygons are reviewed for completion. Verifying every line in the areas of coastal North Carolina densely populated with wetlands is impossible, but the lines are reviewed for completeness. NWI staff again must provide necessary information for any omissions. When the map is approved, technicians ensure projection of the quad to the State Plane Coordinate System. If this has not been completed, the ARC/INFO PROJECT command is employed.

The soils information is prepared in a similar manner to the NWIs, with questions being directed to the county soil scientist. Prior to the steps described previously, soils must be verified for completeness. Because soils are mapped by county boundaries and DCM maps by quad, some files must be joined in quads that intersect county boundaries. At this time, the quad must be checked for differing abbreviations between counties. Discrepancies are handled on a case-by-case basis. When an abbreviation describes different soils in different counties, a temporary abbreviation is created for one of the counties. If a single soil is described by two abbreviations across counties, both abbreviations are incorporated into the classification scheme.

The Landsat data do not require additional verification. Review of this layer is often helpful, however, to ensure that the geographic boundaries match. Cases where landforms do not appear to match require investigation of the discrepancies. If the area is misregistered, this layer might be omitted from the analyses. To date, no area has been mapped without this imagery.

The hydrogeomorphology of a wetland is unique in defining the wetland's function (7). Because these maps serve as the base for additional wetland projects (as described later in this report), an accurate determination of this characteristic is essential. Prior to the overlay procedure, technicians add a new item, hydrogeomorphic (HGM), to the NWI coverage. Because DCM considers both vegetation and landscape position in its classification (discussed later), riverine, headwater, and depressional wetland polygons are assigned an HGM of 'r,' 'h,' or 'd,' respectively. The digital line graphs (DLGs) of hydrography are essential in this step of the procedure.

All wetlands that are adjacent to streams or rivers are considered in the riverine HGM class and are designated as riverine polygons. This class should include all bottomland hardwood swamps and some swamp forests. It rarely includes any of the interfluvial wetland types. If it does, it is a small section of a large interfluvial flatwood from which a small stream emerges. Only the polygons adjacent to the stream are considered riverine. Headwaters are defined as linear areas adjacent to riverine areas that do not have a stream designated on the hydrography data layer. Because these unique systems form the transition between flatwoods and riverine wetlands, they are treated specially. Finally, polygons that exist on interfluvial divides are designated as flats or depressional wetlands. This class should not include any wetlands along streams.

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The complete data coverages are overlaid to create a new, integrated coverage that often approaches 100,000 polygons. Each polygon has many characteristics assigned to it, including the Cowardin classification assigned by the NWI, the soil series provided by the detailed soil lines, the unfiltered land use/land cover code, the filtered land use/land cover code from the Landsat TM imagery, and the HGM classification assigned in the previous step.

Based on these characteristics, each polygon is assigned to one of DCM's classes through an automated ARC/INFO model using an arc macro language (AML). Personnel from the NWI and the North Carolina Department of Environment, Health, and Natural Resources Division of Soil and Water Resources have reviewed the classification of the Cowardin types into DCM wetland types. The classes that DCM currently recognizes are upland, salt/brackish marsh, estuarine shrub scrub, estuarine forest, maritime forest, pocosin, bottomland hardwood, swamp forest, headwater swamp, hardwood flatwoods, piney flatwoods, and managed pinelands. DCM also classifies soils as hydric or nonhydric based on List A of the U.S. Soil Conservation Service (SCS) List of Hydric Soils.

The base of the map is the NWI polygon coverage. Some NWI polygons are omitted from the DCM maps because they are temporarily flooded, but on nonhydric soils or because recent TM imagery indicates these areas are currently bare ground. The managed pineland wetland group on DCM maps includes areas that NWI considers uplands, identified as pine monocultures on the imagery, and that occur on hydric soil.

In addition, DCM also provides a modifier to some of these polygons. DCM notes if NWI has determined that the area has been drained or ditched. Areas designated as wetlands at the time of the NWI photography that currently appear as bare ground on the TM imagery are designated as 'cleared' on the maps. Many of the cleared areas would no longer be considered jurisdictional wetlands. These modifiers are useful indicators of the impacts wetlands sustain from human activities.

Initiation of an interactive session follows completion of the automated procedure. This session considers landscape characteristics that are not easily described to a computer model in correcting the classification. This is especially important in distinguishing bottomland hardwood swamps from hardwood flats. Both contain deciduous, broad leaf species of trees and can be temporarily flooded. The hydrology of these systems, however, is completely different. All bottomland hardwood swamps, for example, must be adjacent to a river where they receive seasonal floodwaters from the channel. Conversely, hardwood flatwoods should be located on interfluvial flats and not adjacent to any streams. Water is not introduced into hardwood flatwoods via a channel; rather, precipitation and ground water provide the water for this system. Polygons that are adjacent to rivers or estuaries but do not have a distinct channel designated in the hydrography coverage are considered headwater swamps.

During the course of methodology development, staff members visited at least 371 sites in the field. As staff members encountered new Cowardin classes, they would verify that the polygons were being placed into the correct DCM categories. If they determined that a particular Cowardin class was systematically misidentified, they updated the algorithm for automation. While this method does not provide for a usable accuracy assessment, it allowed development of the most accurate methods.

The accuracy of these data is unknown at this time. An accuracy assessment of the data is anticipated in the near future. This assessment will allow map users to understand the strengths and limitations of the data. It also will provide an overall summary of data error.

Functional Assessment of Wetlands

Certain initial considerations shaped the approach and methods used in developing a wetlands functional assessment procedure. The procedure needed to fit within the context and objectives of the Wetlands Conservation Plan for the North Carolina coastal area as described above. This context, and the opportunities and limitations it imposed, had considerable influence on the specific procedure developed.

Because we are dealing with a large geographic area with many wetlands, we recognized from the outset that we needed a method we could apply to large land areas without site visits to each individual wetland. This ruled out the many site-specific functional assessment methods that were applied in other contexts. Almost of necessity, a GIS-based approach was chosen. That meant we would have to use information available in GIS format and make use of GIS analytical techniques. The wetland mapping on which the functional assessment is based was performed using GIS, so the basic digital data were available.

The primary objective was to produce information about the relative ecological importance of wetlands that would be useful for planning and overall management of wetlands rather than to serve as the basis for regulatory decisions. While we could not visit every wetland, the goal was to predict the functional assessment value that a detailed, site-specific method would determine. We wanted to be able to predict in advance what the wetland regulatory agencies would determine as a wetland's significance so that the resulting maps would identify those wetlands where a 404 permit would be difficult or impossible to obtain. The resulting information would

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then be useful in determining where not to plan development. This would benefit potential permit applicants by preventing ill-advised plans that would be unlikely to receive permits and simultaneously serve to protect the most ecologically important wetlands. The result of the procedure, then, is not a substitute for a site visit in making regulatory decisions, but a predictor of what a site visit would determine.

A primary consideration was that the procedure be ecologically sound and scientifically valid, based on the best information available about the functions of wetlands. It needed to be based on fundamental principles of wetlands and landscape ecology rather than on arbitrary or subjective decisions.

Finally, the procedure was to be watershed-based. This requirement was primarily because consideration of a wetland's role in its watershed is the soundest basis for determining its ecological significance, but also because the other components of the Wetlands Conservation Plan, including wetland mapping and restoration planning, are based on watershed units. The watersheds being used are 5,000- to 50,000-acre hydrologic units (HUs) delineated by the SCS as illustrated in Figure 1. The North Carolina coastal area comprises 348 of these HUs. Watershed units of any size, however, could be used without changing the validity of the watershed-based considerations used in the procedure.

These initial considerations result in a summary definition of the functional assessment procedure. It is a GIS-based, landscape scale procedure for predicting the relative ecological significance of wetlands throughout a region using fundamental ecological principles to determine the functions of wetlands within their watersheds.

The functional assessment procedure is meant to be used with GIS data for regional application. It is not a field-oriented, site-specific method that involves visiting individual wetlands and recording information. A GIS-based procedure is the only practical approach for dealing with a large geographic area with many wetlands in a limited amount of time.

This GIS-based approach can make information on wetland functional significance available for broad regions in advance of specific development plans. The information is then available for planning to help avoid impacts to the most ecologically important wetlands. In this sense, the North Carolina procedure is unlike other functional assessment techniques that are designed for use in a regulatory context or that require field data for each wetland.

Data Requirements

Because the procedure uses GIS analysis, it requires digital information in GIS format. GIS data layers used in the procedure include:

- Wetland boundaries and types (the topic of the first section of this report).
- Soils maps.
- Land use/land cover.
- Hydrography.
- Watershed boundaries.
- Threatened and endangered species occurrences.
- Estuarine primary nursery areas.
- Water quality classifications.

In the North Carolina coastal area, these data layers either already existed and were available from the CGIA or were developed as part of the Wetlands Conservation Plan. Because other projects funded most of the data acquisition and digitization, developing the necessary GIS databases was not a major cost.

The soils coverage consists of digitized, detailed county soils maps produced by SCS and digitized by CGIA. The soils coverage allows identification of the soil series underlying a wetland, and the properties of the series are used to determine soil capacity for facilitating the wetland's performance of various functions.

The land use/land cover data layer was produced for the APES from interpretation of satellite TM imagery (8). It is used to determine land cover and uses surrounding each wetland and in the watershed.

The basic hydrography coverage consists of 1:24,000-scale USGS DLGs. Because the functional assessment procedure uses stream order as an indicator of watershed position, stream order according to the Strahler system was determined manually and added to the DLG attribute files.

As described previously, the watersheds used in the procedure are relatively small HUs delineated by SCS. DCM contracted with CGIA to have these boundaries digitized for the coastal area. During the digitization process, the watershed boundaries were rectified to USGS and DEM boundaries of larger subbasins to ensure that the HUs could be combined into larger watershed units.

A data layer produced by the North Carolina Natural Heritage Program is used to identify threatened and endangered species occurrences. The North Carolina Division of Marine Fisheries maintains the coverage of primary nursery areas, and the Division of Environmental Management developed a map of water quality classifications that was digitized by CGIA.

The ways in which these data layers are used to determine values for various parameters in the functional assessment procedure are described later in this report. The GIS procedures have been automated using ARC/INFO

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AML on a Sun workstation. The AML programs are available from DCM to anyone planning to use the procedure elsewhere.

Because the assessment procedure was designed for GIS analysis, the choice and expression of individual parameters have been shaped to some extent by the GIS data available and the capabilities and limitations of ARC/INFO techniques and AML automation. DCM was fortunate to have a relatively large amount of GIS data readily available. For use in other areas, the procedure could be modified to use different GIS coverages. At least the first five databases listed above, however, are essential to its basic propositions.

Classification Considerations

The HGM classification system for wetlands (7) classifies wetlands into categories based on landscape position (geomorphic setting), water sources, and hydrodynamics (direction of water flow and strength of water movement). It is being increasingly used as the basis for wetland classification and functional assessment systems. HGM classification focuses on the abiotic features of wetlands rather than on the species composition of wetland vegetation as do most traditional wetland classification schemes.

Several features of the HGM classification system make it a useful starting point for an assessment of wetland functions. Because the HGM system is based on geomorphic, physical, and chemical properties of wetlands, it aggregates wetlands with similar functions into classes. The HGM class of a wetland, in itself, indicates much about the ecosystem functions of the wetland. The HGM approach also forces consideration of factors external to the wetland site, such as water source. This helps relate the wetland to the larger landscape of which it is a part and puts consideration of the wetland's functions in a landscape and watershed context.

Three HGM classes are used as the starting point for the North Carolina functional assessment procedure. All wetlands are first classified as one of the following:

- Riverine
- Headwater
- Depressional

Riverine wetlands are those in which hydrology is determined or heavily influenced by proximity to a perennial stream of any size or order. Overbank flow from the stream exerts considerable influence on their hydrology. Headwater wetlands exist in the uppermost reaches of local watersheds upstream of perennial streams. Headwater systems may contain channels with intermittent flow, but the sources of water entering them are precipitation, overland runoff, and ground-water discharge rather than overbank flow from a stream. Depressional wetlands, including wet flats and pocosins, generally are not in direct proximity to surface water. While they may be either isolated from or hydrologically connected to surface water, the hydrology of depressional wetlands is determined by ground-water discharge, overland runoff, and precipitation.

The functions of wetlands in these different HGM classes differ significantly. Riverine wetlands regularly receive overbank flow from flooding streams and, thus, perform the functions of removing sediment and pollutants that may be present in the stream water and providing temporary floodwater storage. Headwater and depressional wetlands cannot perform these functions because they do not receive overbank flow. Headwater wetlands occur at landscape interfaces where ground water and surface runoff coalesce to form streams. Headwater wetlands provide a buffer between uplands and stream flow so they can perform significant water quality and hydrology functions. While depressional wetlands do not perform buffer functions, they often store large amounts of precipitation or surface runoff waters that otherwise would more rapidly enter streams. Wetlands in all HGM classes can perform important habitat functions.

Because the wetlands in these different HGM classes are functionally different, their functional significance is assessed using different, though similar, procedures. If the same procedure were used for all HGM classes, depressional wetlands would always be considered of lower functional significance simply because they are not in a landscape position to perform some of the water quality and hydrologic functions of riverine and headwater wetlands.

In addition to HGM classes, wetland types identified by dominant vegetation are used at several points in the functional assessment. This reflects a recognition that the biologic properties of a wetland site considered together with its hydrogeomorphic properties can provide a more detailed indication of its functions than either taken alone. The HGM class of a wetland, as a broad functional indicator, determines which assessment procedure to use. Within each HGM class and corresponding assessment procedure, wetland type determines the level or extent of specific parameters.

The wetland types used are those typical of the North Carolina coastal area. They result from a clumping of the Cowardin classes used on NWI maps into fewer types with more intuitively obvious type names (e.g., swamp forest, pocosin), as described previously. These wetland types are used in the wetland maps that form the starting point for the functional assessment.

Wetland types are used in the procedure as indicators of functional characteristics. Correlations between wetland type and wetland functions were determined from

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statistical analysis of field data from nearly 400 sites. At each site, the presence or absence of a list of functional indicators was recorded. Dr. Mark Brinson of East Carolina University developed the functional indicators lists, in part. Dr. Brinson served as primary scientific consultant in developing the HGM classification system and the field sampling methodology.

Wetland types differ in other areas, so their inclusion in this procedure limits its use in its current form to the southeastern coastal plain. Adaptation of the procedure for use in other areas would require either extensive field sampling as was performed in coastal North Carolina or a more arbitrary clumping of wetland types based on best professional judgment. Other methods of wetland classification could be used, provided wetlands are classified in such a way that functional characteristics of the wetland types are constant and can be determined by field sampling, literature values, and/or professional judgment. The procedure could be applied directly to NWI polygons if these are the only wetland map base available.

In addition to wetland type, several other parameters are used as indicators of the existence or level of specific wetland functions. These include both site-specific parameters, such as wetland size and soil characteristics, and landscape considerations, such as watershed position, water sources, land uses, and landscape patterns. GIS analysis determines values for these parameters based on the data layers discussed above. They could be determined manually, but the process would be very labor intensive.

Unlike assessment procedures that depend solely on information that can be collected within a wetland, this procedure relies heavily on factors external to the wetland site itself. Relationships between a wetland and the landscape within which it exists are integral considerations in determining wetland functional significance. Characteristics of the landscape surrounding a wetland are often more important determinants of its functional significance than are the characteristics of the wetland itself. Of the 39 parameters evaluated in the procedure, 21 are landscape characteristics, and 18 are internal characteristics of the wetland itself.

While we believe this emphasis on a wetland's landscape context is a more ecologically sound approach to functional assessment than site-specific methods, it requires a great deal more information than could be collected within the wetland itself. The procedure is based on GIS data and analysis, not only to make it suitable for regional application, but because GIS provides the most practical way to analyze the spatial relationships of landscape elements and their properties.

Structure of the Assessment Procedure

The assessment procedure uses a hierarchical structure that rates individual parameters and successively combines them to determine the wetland's overall functional significance. The complete hierarchical structure is illustrated in Figure 2. It consists of four levels:

- Overall functional significance of the wetland.
- Specific functions and risk of wetland loss.
- Subfunctions.
- Parameters evaluated to determine the level and extent of functions.

The objective of functional assessment is to determine an individual wetland's ecological significance in its watershed and the larger landscape. The highest hierarchical level, or end result of applying the procedure, then, is the wetland's overall functional significance.

The second hierarchical level includes the four primary factors that are considered in determining the wetland's functional significance (see Figure 3). The overall ecological significance of a wetland is determined by the degree to which it performs, or has the capacity to perform, specific functions. The broadest grouping of wetland functions includes water quality functions, hydrologic functions, and habitat functions. The nature of the landscape and the water characteristics of the watershed in which a wetland functions also determine ecological significance to some extent. These factors determine the potential risk to watershed and landscape integrity if the wetland functions were lost. Including a "risk factor" as a basic consideration in functional assessment also provides a means of considering cumulative impacts and the practicality of replacing lost functions through mitigation in determining a wetland's overall significance.

Each primary function of wetlands is actually a combination of separate, more specific subfunctions. Water quality subfunctions include the removal of nonpoint source pollutants from surface runoff and the removal of suspended or dissolved pollutants from flooding streams. Hydrology subfunctions include storage of precipitation and surface runoff, storage of floodwater from streams, and shoreline stabilization. Habitat subfunctions include providing habitat for both terrestrial species and aquatic life. Several considerations that, while not truly wetland functions, are called subfunctions for parallelism also determine risk factor. The subfunction levels of the assessment procedure are illustrated in Figures 4 through 7.

Properties of the wetland and its surrounding landscape determine the extent to which a wetland performs these different subfunctions. The assessment procedure refers to these properties as "parameters." Parameters

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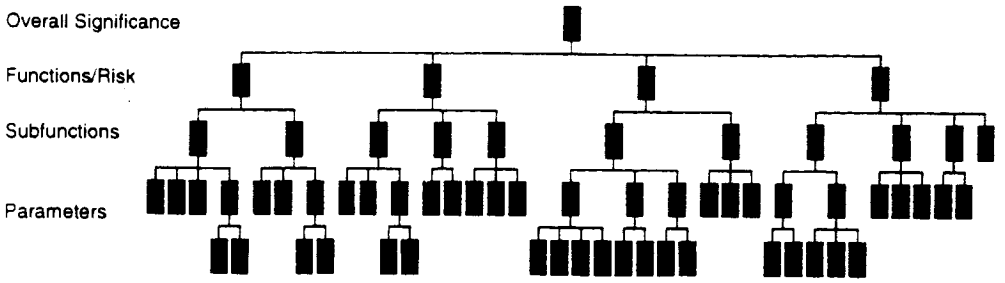

Figure 2. Overall hierarchical structure of the functional assessment procedure.

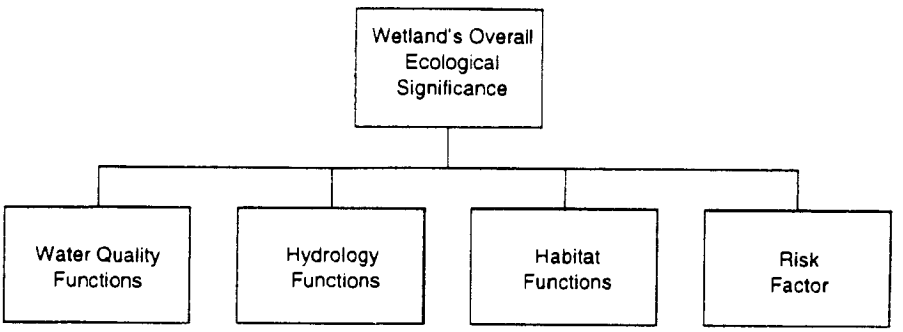

Figure 3. Assessment level two: Primary wetland functions and risk factor.

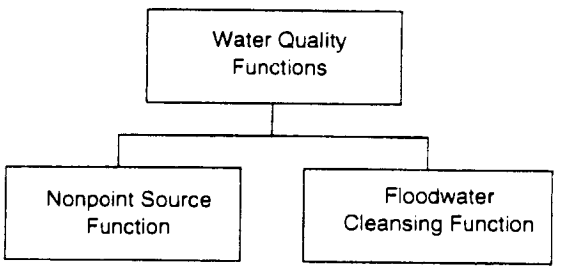

Figure 4. Water quality subfunctions.

make up the levels in the hierarchical structure that are actually evaluated based on fundamental ecological considerations. Parameter values, in turn, are combined to produce ratings for the subfunctions. Future reports will explain in detail all parameters evaluated in the assessment procedure and document them for scientific validity. This paper discusses only the parameters under the nonpoint source removal subfunction of the water quality function for illustration (see Figure 8).

The first parameter determining a wetland's significance in removing nonpoint source pollutants from surface runoff water is whether the water contains sediment, nutrients, or toxic pollutants in significant quantities. This is evaluated in the "proximity to sources" parameter based on the land uses surrounding the wetland. If agricultural fields or developed areas from which pollutants are likely to enter surface runoff largely surround the wetland, the wetland's potential for removing nonpoint source pollutants is high. If, on the other hand, natural vegetation from which runoff water is likely to be largely unpolluted mostly surrounds the wetland, its potential for removing significant pollutants is low.

Proximity to sources is an "opportunity" parameter. That is, it determines whether a wetland has the opportunity to remove pollutants from surface runoff by considering how likely the runoff water is to be polluted. The other parameters for this subfunction are "capacity" parameters that measure the wetland's ability to perform the function if the opportunity is present. Opportunity and capacity parameters are treated differently in determining a wetland's overall significance to prevent a wetland from being rated lower simply because present opportunity does not exist. This is discussed in more detail below.

The second parameter considered in determining a wetland's significance in nonpoint source removal is its proximity to a surface water body. If runoff entering a wetland would otherwise directly enter surface water, the wetland's significance as a filter is greater than if the wetland is far removed from surface water. In that case, pollutants in runoff could either settle out or be removed by other means before they enter surface water as pollutants.

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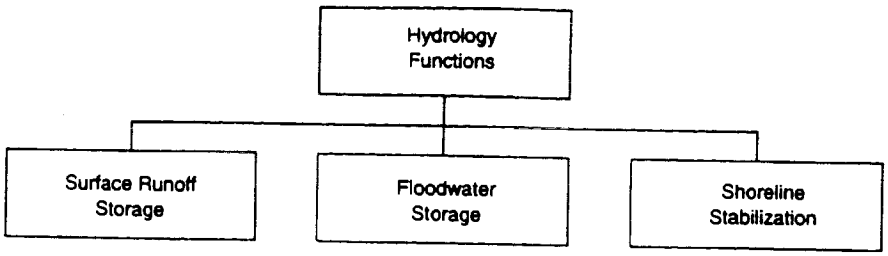

Figure 5. Hydrology subfunctions.

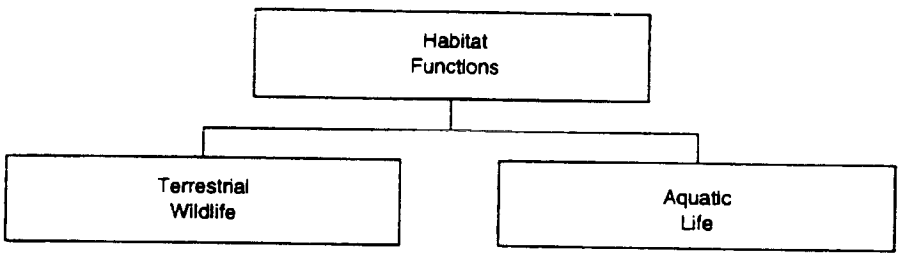

Figure 6. Habitat subfunctions.

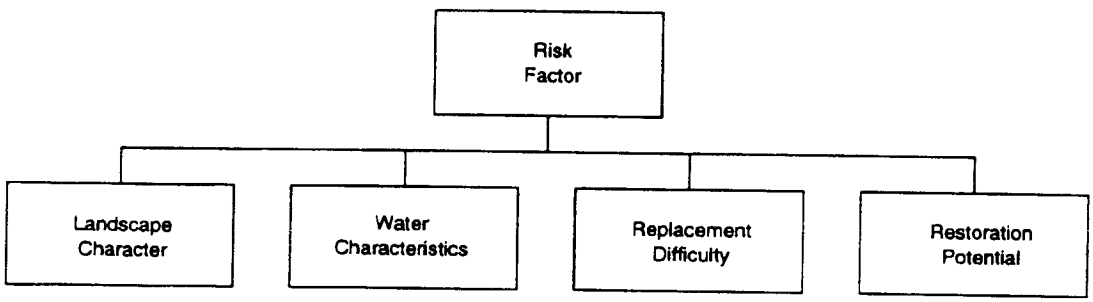

Figure 7. Risk factor subfunctions.

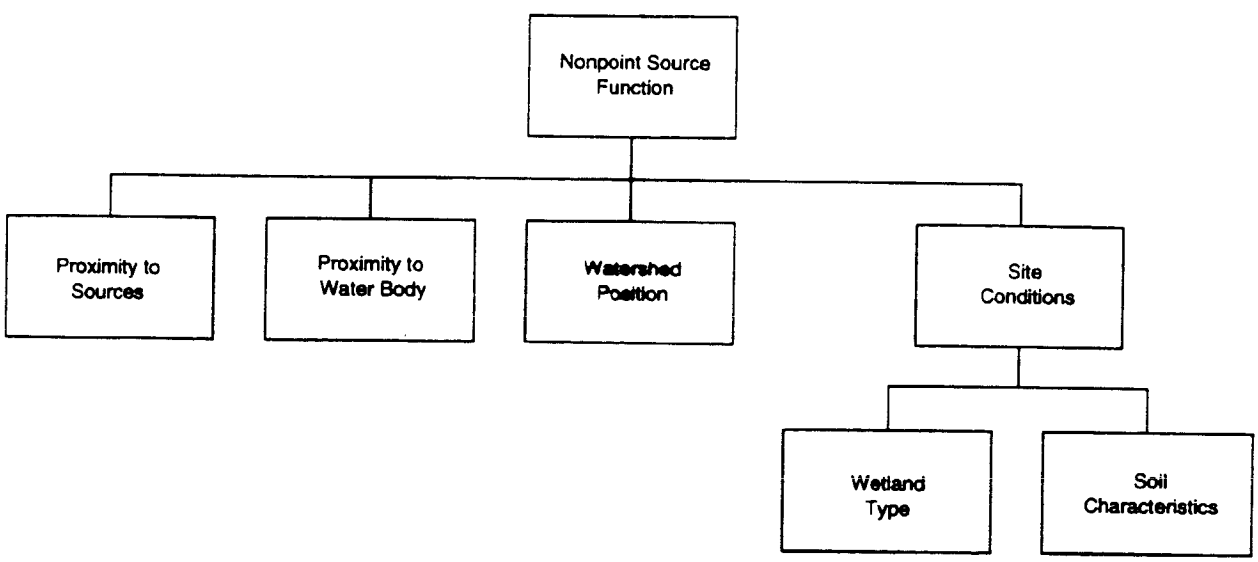

Figure 8. Parameters evaluated under nonpoint source pollutant removal subfunction.

The third parameter is the position of the wetland in its watershed. Several studies have documented that headwater wetlands are most effective in removing nonpoint source pollutants (9-11). Thus, the higher in its watershed a wetland is located, the higher is its significance in nonpoint source removal.

Two subparameters, wetland type and soil characteristics, determine the value of the fourth parameter, site conditions. By virtue of their typical microtopography, hydrology, and vegetative structure, some wetland types more effectively retain and filter surface runoff than do other types. Some soil series are more effective than others in retaining and chemically transforming pollutants. Each subparameter is rated, and their combined values produce a rating for the site conditions parameter.

A similar evaluation of specific parameters is performed to derive significance ratings for other wetland subfunctions. In all cases, GIS analysis determines parameter values based on the data layers described above. Some parameters, such as wetland type in the nonpoint source illustration, are surrogates or indicators of other wetland properties that actually determine the wetland's functional capacity. The limitations of GIS data and techniques necessitate the use of indicator parameters.

Evaluation Procedure

The objective of the assessment procedure is to determine an individual wetland's ecological significance in the watershed in which it exists. Ecological significance is divided into three broad classes (high, medium, and low) rather than attempting to derive a specific numerical "score." This is partly because of the procedure's initial application in an EPA ADID project performed by DCM in Carteret County, North Carolina. Standard ADID procedure is to classify wetlands into three groups:

- Areas generally unsuitable for the discharge of dredged or fill material.
- Areas that require a project-by-project determination.
- Possible future disposal sites for dredged or fill material.

These groups correspond to the H, M, and L used in the assessment procedure.

The approach of classifying wetlands into three broad functional significance classes is also used, however, because it is feasible with our current understanding of wetland function. Attempting to assign a specific value along a numeric continuum of functional significance greatly exaggerates the precision with which we can realistically apply current knowledge. The three significance classes used in the assessment procedure provide the information necessary to meet the procedure's objectives without going beyond the realm of reasonable scientific validity.

As explained above, the basic evaluation is performed at the parameter level. An H, M, or L value is assigned to each parameter as it relates to the performance of the wetland subfunction being considered. For example, if the soils underlying a wetland have properties that are highly conducive to the function being considered, the soil characteristics parameter is rated H; if soil properties are less conducive to performing the function, the parameter is rated M; and if soil properties are not at all conducive to the function, the parameter is rated L. All individual parameters under a given subfunction receive similar ratings.

The individual parameter ratings are then combined to give an H, M, or L rating for each subfunction. The subfunction ratings are combined into a rating of the wetland's significance in performing each of the primary wetland functions. Finally, the ratings for primary functions are combined into an overall rating of the wetland's functional significance.

The process of successively combining ratings up the structural hierarchy is the most complex aspect of the assessment procedure. The combining, as well as the evaluation of individual parameters, is based on fundamental ecological principles about how wetlands and landscapes function. Because the ecological processes themselves interact in complex ways, combining ratings is much more complex than a simple summation of individual ratings. Some parameters are normally more important than others in determining the level at which a wetland performs a specific function and, thus, must be weighed more heavily in determining the combined value. In some cases, different combinations of individual parameter ratings result in the same level of functional significance. Each possible combination of parameters must then be considered.

The automated version of the assessment procedure maintains all individual parameter ratings and combinations in a database. Because the combining process is complex, the reason a wetland receives an overall H, M, or L rating may not be intuitively obvious. The database makes it possible to trace through the parameter, subfunction, and primary function ratings that result in a wetland's overall rating.

This database also allows consideration of specific wetland functions individually. For example, in a watershed targeted for nonpoint source pollution reduction, one management objective may be to give the highest level of protection to wetlands most important in performing this function. The database allows examination of each wetland for its significance in nonpoint source removal and production of a map of wetlands rated according to their significance for this single function.

Individual function ratings in the database can also be used to improve planning, impact assessment, and

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mitigation for development projects that affect wetlands. If alternative sites are available, such as alternative corridors for a highway, the alternative with the least impact on the wetland function considered most important in the watershed can be identified. Rather than simply minimizing acres of wetland impact, the objective would be to minimize impacts to the most important wetland functions. Environmental assessment of wetland impacts can identify specific functions to be lost. Mitigation can be improved by giving priority to sites with the highest potential for performing the same functions.

Future reports will explain detailed procedures for evaluating individual parameters and combining them into functional ratings. This paper illustrates only the water quality nonpoint source removal subfunction. The rating system for this subfunction is summarized in Figure 9.

Four parameters are evaluated to determine the significance of the nonpoint source removal subfunction. Because (d), the site conditions parameter, has subparameters below it, it is first evaluated using a relatively simple procedure. If conditions typical of the wetland type and characteristics of the underlying soil are both highly conducive to removal of pollutants in runoff water entering the wetland, the site conditions parameter is H. If either the wetland type or the soil is not at all conducive to pollutant removal and the other subparameter is no more than somewhat conducive, the site conditions parameter is L. Any other combination results in an M.

Parameters

(a) Proximity to Sources

(b) Proximity to Surface Water

(c) Watershed Position

(d) Site Conditions

(1) Wetland Type

(2) Soil Characteristics

Evaluation Procedures

Site Conditions

H Both Parameters H

M Other combinations

L One parameter L and neither H

NPS Subfunction

H (a) & (b) H and (d) at least M *or*
(c) & (d) H and (b) at least M

M Other combinations

L Two of (b), (c), & (d) L

Figure 9. Parameters evaluated under nonpoint source pollutant removal subfunction.

Following evaluation of all parameters, they are combined to evaluate the significance of the wetland in removing nonpoint source pollutants. Two combinations result in the wetland being evaluated as highly significant in performing this function. First, if the wetland is adjacent to both a significant source of polluted runoff (a = H) and a permanent surface water body into which the runoff would flow if the wetland were not there (b = H), *and* has site conditions that are at least reasonably efficient in catching, holding, and removing pollutants from the runoff (d at least M), it receives an H. Alternatively, even if the wetland is not adjacent to a pollutant source, it receives an H if it is in the headwaters of the watershed (c = H), site conditions are highly conducive to pollutant removal (d = H), and it is at least close to an intermittent stream (b at least M).

On the other hand, if any two of parameters (b), (c), and (d) are evaluated L, the significance of the wetland for nonpoint source pollutant removal is Low. That is, the wetland is evaluated as L for this function if any of the following conditions exist:

- The wetland is not close to surface water (b = L) and downstream in the watershed (c = L).
- The wetland is not close to surface water (b = L), and its site conditions are poor for pollutant removal (d = L).
- The wetland is downstream in the watershed (c = L) and has poor site conditions (d = L).

Any combination of parameter evaluations other than those resulting in an H or L results in the wetland being evaluated as of moderate significance for removing nonpoint source pollutants. This example is typical of evaluation procedures used for all subfunctions. More often than not, the evaluation procedures are complex and multifarious in their reasoning and application. Hopefully, though, they are scientifically valid based on current knowledge of wetland ecology.

Opportunity and Capacity

The concepts of opportunity and capacity for a wetland to perform a given function were briefly discussed above. For a wetland to actually perform a function, it must have both the opportunity and the capacity for the function. In terms of the nonpoint source example, a source of potentially polluted runoff must enter the wetland to provide an opportunity, and the wetland must have the internal capacity to hold the runoff and remove the pollutants before releasing the water. Factors external to the wetland usually determine the opportunity to perform a function, while properties of the wetland itself along with its landscape position determine the capacity to perform the function.

Because the assessment procedure is a landscape scale procedure that evaluates the functions a wetland

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performs in relation to its surroundings, essentially every subfunction includes opportunity parameters. A functional assessment that is too heavily dependent on opportunity parameters, however, is static and rapidly becomes invalid as land uses change. A wetland that is bordered by natural forest today can be bordered by a young pine plantation or a subdivision under construction by next year. The fact that a wetland does not have the opportunity to perform certain functions today does not mean that it will not have the opportunity in the future. If an assessment of wetland significance is to remain valid over time in a landscape subject to change, opportunity parameters alone cannot be determinative.

The evaluation procedure for the nonpoint source subfunction explained above is an example of how the assessment procedure handles this situation. The opportunity for a wetland to receive polluted runoff water from surrounding lands (a = H) *can* result in an evaluation of H for this subfunction if other properties are also present, but it does not *have* to be present for a wetland to be evaluated H. Other parameters (c and d = H, and b at least M) that give a wetland a high capacity to remove nonpoint source pollutants can also result in an H. Conversely, lack of present opportunity (a = L) does not result in an evaluation of low significance for this function. At least two of the other parameters must be L for the wetland to be evaluated as L.

These conventions hold throughout the procedure. A present high opportunity to perform a function can result in an evaluation of high significance for the function, but high capacity can also result in an H evaluation even if present opportunity is lacking. Lack of present opportunity alone never results in an evaluation of low significance for a function. High opportunity is treated essentially as a "bonus" consideration that can result in a higher evaluation for a wetland than its capacity alone would indicate but that will never result in a lower evaluation because of its absence.

Overriding Considerations

Several considerations are of such importance in the North Carolina coastal area that their presence alone will result in a wetland evaluation of high significance. These parameters are evaluated first as either true or false, and if one or more of them is true, the rest of the evaluation procedure is not performed.

The first overriding consideration is whether the wetland is a salt or brackish marsh meeting the definition of "coastal wetland" as set forth in North Carolina statutes (NCGS 113-229(n)(3)) and rule (NCAC 7H .0205(a)). Coastal wetlands in North Carolina are designated by law as highly significant. Consequently, the assessment procedure evaluates them automatically as H and includes no considerations for differentiating among the functional significance of these wetland types.

The second overriding consideration is whether the wetland is adjacent to an officially designated primary nursery area (PNA). All designated PNAs are included in "areas of environmental concern" in the NC CMP and are protected by a specific set of regulations. They are areas where initial postlarval development of finfish and crustaceans takes place and, thus, are critical to estuarine fish and shellfish populations. Wetlands adjacent to PNAs are highly important in maintaining water quality and appropriate salinity gradients in these critical areas and are automatically evaluated as of high functional significance.

The third overriding consideration is whether the wetland contains threatened or endangered species. If a known threatened or endangered plant or animal species on either federal or state lists is present, the wetland is evaluated as highly significant. The determination is based on information obtained from the North Carolina Natural Heritage Program.

The fourth overriding consideration is whether the wetland includes all or part of a critical natural area as designated by the North Carolina Natural Heritage Program. If so, the site is considered of high significance. GIS data layers maintained by the Natural Heritage Program also help make this determination.

Verification

Throughout the development and initial application of the assessment procedure, we have checked and verified its validity. Parameter evaluations and combination procedures are based on the best wetland science available in the scientific literature. The validity and accuracy of the GIS databases used to apply the procedure have been verified to the extent possible. Following sections of this report fully document any assumptions made about wetland ecology, GIS data, or GIS analytical techniques.

An advisory panel of wetland scientists familiar with the wetlands of coastal North Carolina and representatives of several state and federal wetland-related agencies reviewed every step of the procedure's development. While their review does not represent an endorsement of the procedure or its results by the agencies or individuals included, it does indicate the level of peer review the procedure has received.

During development of the procedure, field visits were made to nearly 400 wetland sites to gather data on functional indicators. On these same site visits, a field-based functional assessment procedure, the Wetland Rating System developed by the North Carolina Division of Environmental Management, was applied. This provides the basis for a field verification of the assessment procedure.

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Discussion

As we continue to understand more about the role of wetlands in maintaining a healthy environment, the usefulness of wetlands locational data continues to grow in importance. Spatial data can assist county planners in guiding development away from environmentally sensitive areas. Landowners now have the capability to look at a map and realize very quickly that wetlands exist on a given area of land. In addition, economic development councils can use this information to plan development in areas attractive to a particular industry. If a new business or industry wishes to locate in an area positioned such that the wetlands permitting process could be avoided, maps showing lands void of wetlands could be a significant tool to the economic development council. The representation of these wetlands' ecological significance dramatically increases the utility for these data.

While paper maps can be distributed to all interested parties, digital data also are available to public agencies who have GIS capabilities. In Carteret County, for example, a publicly installed workstation will be made available with these data installed. The county government will be able to view wetlands in the context of cadastral boundaries that already are on GIS. Information about sensitive resources made available prior to any development will, hopefully, lead development away from environmentally sensitive areas.

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Decision Support System for Multiobjective Riparian/Wetland Corridor Planning

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Kansas has numerous programs that affect riparian corridors and associated wetlands. These programs include planning, monitoring, assistance, research, and regulatory activities. Although administration of these programs often overlaps, integration of program objectives into a holistic, multiobjective approach to resource planning and management has been lacking. A large amount of resource data was routinely collected and compiled, but no effective way had been developed to integrate these data into the decision-making process.

The Kansas Water Office (KWO) was awarded a grant in September 1992 from the U.S. Environmental Protection Agency (EPA) to develop a geographic information system (GIS) decision support system (DSS) that would enable the state to augment its ability to manage riparian/wetland corridors. The project used GIS to differentiate between reaches of a stream corridor to evaluate their environmental sensitivity. The Neosho River basin, one of 12 major hydrologic basins in Kansas, was used as a pilot to demonstrate the feasibility of the concept.

The KWO will use the DSS to help target sensitive areas in the Neosho basin for further planning activities. The project will also benefit other state agencies in their riparian/wetland corridor efforts. The implementation of planning objectives may involve local units of government and, ultimately, private landowners.

Major phases of the project included:

- A needs assessment study
- A feasibility analysis
- A system design
- Construction of the DSS for the Neosho River basin
- A final evaluation of the DSS capabilities

An interagency project advisory group (IPAG), consisting of representatives from eight agencies directly or indirectly involved in riparian and wetland protection activities, was formed to assist in project design and evaluation.

Major steps involved in designing the DSS included:

- Selection and GIS development of databases used for riparian corridor evaluation.
- Creation of riparian corridor segments.
- Development of an analysis methodology to apply to corridor segments.
- Evaluation of the DSS.

Databases Selected for Decision Support System Development

Many types of data were reviewed for the DSS. Several were not used due to the costs associated with geographically referencing the data, given the current data format.

The databases listed in Table 1 are available in the DSS.

During the system design phase of the project, the IPAG identified the need to develop a pilot study area for the DSS. The IPAG had difficulty understanding how a DSS would use geographically referenced data sets (coverages). Before committing to a design for the development of a basinwide system, the IPAG decided first to develop a pilot study area, with a specific focus (application), that could be on-line and demonstrated early. This would allow time for further refinement of the scope of work and identification of coverages to be developed prior to basinwide development of the DSS. For the pilot study application, the IPAG chose to assess the value and vulnerability of the riparian areas in two 11-digit hydrologic unit code (HUC11) watersheds to allow the user to evaluate a corridor segment and compare between segments and to prioritize or target segments for further planning activities.

As development of data layers progressed for the pilot, the IPAG quickly determined that the DSS project

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Table 1. DSS Database List

DSS Name	Data Description	Source
Boundary	Neosho River basin boundary	Soil Conservation Service (SCS) HUC11 drainage basins; 1:100,000-scale[a]
Buffer	Riparian corridor	Original buffer on mainstem Neosho and Cottonwood; 147 corridor segments split on tributary confluences
Channels	Stream channelization	Division of Water Resources (DWR) legal description of locations
Con_ease	Conservation easements	Locations of important natural resources that could be purchased by the state from willing landowners for conservation protection
Contam	Water contamination	Kansas Department of Health and Environment (KDHE) contamination locations[a]
Corridor	Riparian corridor	Final riparian corridor; 63 corridor segments developed from HUC11 boundaries
County	County boundaries	Kansas Geological Survey (KGS) cartographic database; 1:24,000 scale[a]
Dams	Dam structures	DWR legal descriptions of locations
Dwrapp	Water appropriations	DWR legal descriptions of locations[a]
Gages	United States Geological Survey (USGS) stream gaging stations	USGS latitude-longitude descriptions; GIS cover developed by USGS
Geology	Surface geology	KGS 1:500,000-scale[a]
Huc11	11-digit hydrologic unit boundaries	SCS HUC11 drainage basins; 1:100,000-scale[a]
Hydr100k	Hydrology	USGS 1:100,000-scale digital hydrology[a]
Kats	Kansas water quality action targeting system	KDHE target valuable and vulnerable scores by HUC11 drainage basin
Landc	Land cover	1:100,000-scale developed from satellite imagery by the Kansas Applied Remote Sensing Program, University of Kansas[a]
Lc_stats	Land cover statistics	Summary statistics on land cover by corridor segment
MDS	Minimum desirable stream flow monitoring gages	Subset of USGS gaging stations
NPS	Nonpoint source pollution	Target watersheds identified in the Kansas Water Plan
Perenial	Perennial hydrology	Reselected perennial streams from 1:100,000 USGS digital hydrology
Pop	Population	Urban land cover (from landc) with 1980 and 1990 Census population data
PPL	Populated places	Geographic names information system (GNIS) entries for Kansas; GIS cover developed by USGS
Publand	Public lands	State and federally owned land digitized from 1:100,000-scale USGS quad maps
Roads	Roads	USGS 1:100,000-scale digital roads[a]
Sections	Section corners	KGS cartographic database; 1:24,000-scale[a]
Streamev	1981 stream evaluation	U.S. Fish and Wildlife stream evaluation study; Kansas Department of Wildlife and Parks (KDWP) provided data on paper maps
T_and_e	Threatened and endangered species	Stream locations of state and federal identified threatened and endangered species; KDWP provided data on paper maps
Temussel	Threatened and endangered species	Locations of state endangered floater mussels; KDWP provided data on paper maps
Tigrcity	City boundaries	U.S. Census 1 100,000-scale TIGER line data; boundaries only, areas not named (use with PPL)
Twp	Townships	KGS cartographic database; 1:24,000-scale[a]
Watrfowl	Water fowl locations	KDWP locations and counts of annual waterfowl migration; data developed from paper maps (Restrict public distribution of data per KDWP request.)
Wq_eff	Water quality: effluent	KDHE sampling sites; GIS cover developed by KDHE
Wq_grnd	Water quality: ground water	KDHE sampling sites; GIS cover developed by KDHE
Wq_lake	Water quality: lake	KDHE sampling sites; GIS cover developed by KDHE
Wq_strm	Water quality: stream	KDHE sampling sites; GIS cover developed by KDHE

[a] Data available at the Kansas Data Access and Support Center (DASC).

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parameters would have to be limited to the riparian corridor along the mainstem of the Neosho and Cottonwood Rivers. The costs associated with developing riparian corridor segments for all perennial waters in the Neosho basin was far greater than the available funding.

Creation of Riparian Corridor Segments

A buffer width of one-half mile (one-quarter mile from each stream bank) for the mainstem of the Neosho and Cottonwood Rivers was used to produce the riparian corridor. If more time and funding had been available, riparian corridors for all perennial streams in the Neosho basin could have been developed. The development of this second view of data, organized by the HUC11 watershed, would then have been useful for individual watershed analysis because all perennial streams in the watershed could be analyzed.

The intersection of the HUC11 basin boundaries segmented the corridor. In several instances, small sliver polygons were produced where the HUC11 boundary paralleled the river within the 1/4-mile corridor. The sliver polygons were dissolved into the majority HUC11. In other words, this project assumed that the 1/4-mile corridor buffer was more accurate and useful than the 1:100,000-scale HUC11 boundary.

Many of the HUC11 boundaries that the Soil Conservation Service (SCS) developed actually follow the course of the Kansas streams, rather than intersect them. When this occurred along the Neosho and Cottonwood Rivers, we found that the resulting opposing corridor segments did not always balance with an equivalent length. Also, some HUC11 boundaries would first follow the river, then cross the river. This resulted in corridor segments that encompass both sides of the river for a portion of the segment and follow only one side of the river for another portion of the segment. To address these situations, the KWO arbitrarily added intersections to create equivalent left and right bank corridor segments and to create corridor segments that encompassed either one side of the river or both sides of the river.

Once the corridor segments were finalized and numbered, the corridor segment identification number (corrseg-id) was attached to the other GIS covers. This allows the reselection of data for a given corridor segment, using Boolean expressions in the DSS.

Development of an Analysis Methodology: Land Use

The IPAG determined that one of the most significant factors associated with the quality of the riparian corridor is land cover. Land cover was analyzed for the riparian corridor segments; the GIS cover lc_stats contains summary statistics for each corridor segment. The calculations discussed in the following paragraphs identify the data found in the lc_stats cover. Due to the size of the land cover data set in the Neosho River basin, the DSS includes only the land cover within the riparian corridor.

One way of identifying corridor segments in need of protection or remedial action is to determine the ratio of the number of acres in the corridor segment that contain the preferred riparian land cover types (grasses, woods, and water) to the number of acres that contain the least preferred types of land cover (crops and urban areas). The corridor segments can then be ranked according to that ratio.

Other calculations are useful:

- *bad_pct:* percentage of the corridor segment that contains crop and urban land cover types.
- *bad_tbad:* percentage of all crop and urban land cover for the entire riparian corridor that resides in the corridor segment.
- *'type'_pct:* percentage of the corridor segment that is crop, grass, wood, water, and urban. 'Type' refers to each of the five land cover types; lc_statsuses: a separate value for each (e.g., crop_pct).
- *'type'_t'type':* percentage of each type of land cover for the entire riparian corridor that resides in the corridor segment (e.g., crop_tcrop).
- *'type'_acres:* total acreage of each type of land cover in the corridor segment (e.g., crop_acres).
- *good_acres:* total acreage of grass, wood, and water in the corridor segment.
- *bad_acres:* total acreage of crop and urban in the corridor segment.

Another significant benefit of the DSS is the ability to see where the land cover types are in relation to the river. As an example, the ability to identify corridor segments that have crop land extending to the river on both banks is useful because they are the segments most vulnerable to bank erosion. Those segments can then be targeted for further remedial activities planning.

Decision Support System Requirements

The DSS data sets were developed and analyzed using ARC/INFO on a UNIX-based workstation. The final covers were then exported and transferred to a microcomputer for use in ARC/VIEW. Hardcopy prints are printed to a Tektronix Phaser III color wax printer with 18 Mb of RAM, running in Postscript mode.

The DSS data sets total 26 Mb. ARC/VIEW version 1 requires 8 Mb of RAM to load the program. To run the DSS efficiently, a 486DX-66 with 16 Mb of RAM is preferred. The DSS is slower on a 486DX-33 with 8 Mb of RAM. It was not tested on any other PC configuration, so a configuration in between the two may be satisfactory.

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Processing GIS Data

Reselecting the perennial streams in the Neosho basin and further identifying the mainstem of the Neosho and Cottonwood Rivers using United States Geological Survey (USGS) 1:100,000-scale hydrography can be time consuming. Perhaps the River Reach III covers should replace that data in the future.

Attaching census data to the urban land cover polygons, as was done for the pop cover, is not recommended. Use of the TIGER line files and cover would give a more accurate distribution of the population. Because less than 5 percent of the riparian corridor had urban land cover, the KWO did not use the pop cover in its evaluation. Several summary covers of TIGER and census data will soon be available from DASC.

Clipping the other ARC/INFO covers to the Neosho basin and attaching the corrseg-id, using the identity command, was unremarkable.

Processing Non-GIS Digital Data

Channels and dams were in digital format but were not in ARC/INFO format. The files were processed using the LeoBase conversion software from the KGS, then generated into ARC/INFO covers. Some records were lost in the conversion. The LeoBase program fails to convert, or incorrectly converts, legal descriptions for sections that do not have four section corners (e.g., northeastern Kansas). The Division of Water Resources is in the process of attaching latitude-longitude to the point locations. Processing these data should take only a few hours at most.

Processing Nondigital Data

Several covers were developed on contract from paper maps or legal descriptions. They were: conservation easements (con_ease), public lands (publand), stream evaluation (streamev), threatened and endangered species (t_and_e and temussel), and water fowl (watrfowl). Most of the data for these covers were drafted on a 1:100,000-scale USGS quad map and digitized. The stream evaluation data were developed using a scanned paper map of the coded streams as a backdrop for the 1:100,000-scale hydrography; the digital streams were reselected and coded.

In summary, KWO's GIS personnel needed approximately 275 hours to develop the riparian corridor segments, process the land cover data and summary statistics, export the covers, transfer and import the covers for ARC/VIEW, and assist in the development and presentation of the DSS demo. Contract personnel spent approximately 183 hours developing GIS covers for the DSS. This does not include the time spent identifying the perennial and mainstem hydrology in the USGS 1:100,000-scale hydrology.

Final Evaluation of the Decision Support System

In its final evaluation of the system, the IPAG determined the system to be useful and an excellent start at consolidating a variety of data that have application for riparian corridor/wetland issues. Many IPAG members found ways to use the DSS in their own programs. Additional comments on the system evaluation are as follows:

- Concern about the lack of complete wetland data. The land cover data available could not identify wetland areas.
- Need for more detailed woodland data. Again, the resolution of the land cover data precluded detailed identification of woodland areas. The Kansas Biological Survey (KBS), the KWO, and EPA are now pursuing options to develop more detailed land cover data, including wetlands and woodlands.
- The lack of information on the tributaries did not allow full basin analysis, which would be desirable. This issue is addressed in the "construction" discussion above.
- Desirability of expanding the project with elevation and temporal data.
- Lack of definition of the floodplain. Federal Emergency Management Agency (FEMA) floodplain data are not easily incorporated into a GIS. Other options, including satellite imagery of the flood of 1993, will be evaluated.
- Project development requires extensive communication between program people and GIS technicians. This can be a daunting task due to the technical vocabularies involved and the many other ongoing activities of the participants.
- Consideration of the requirements for transferring the project to other potential users. GIS applications generally use large databases. User microcomputers may not have the CPU, RAM, and storage capacity necessary for the DSS application and often have a limited number of options for data transfer.
- Concern about costs and time associated with the expansion of the DSS to other basins in the state. This project was focused on one of the 12 major hydrologic regions in Kansas. Funding options, project scope, and system refinements based on the physical characteristics of the other basins need to be pursued.

The KWO learned that clearly defining a single DSS application at the outset of the project is critical. The KWO originally believed that the DSS could be developed with general descriptions of the broad range of program applications, utilized by multiple agencies, that could benefit from the DSS. Each participating agency

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could bring its programs and needs to the IPAG for discussion; the resulting DSS would then serve those multiple programs and needs. Instead, the ambiguity of the objective confused the IPAG. Once the IPAG chose to focus on a single application, the assessment of riparian corridor value and vulnerability to target priority areas for further planning activities, the IPAG became more confident in its advisory role. Upon completion of the project, the IPAG members could readily identify how the DSS could be enhanced, modified, or directly used in their own programs.

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Design of GIS Analysis To Compare Wetland Impacts on Runoff in Upstream Basins of the Mississippi and Volga Rivers

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Introduction

The attention given in hydrologic studies to wetlands differs significantly between the United States and Russia at the present time. Fundamental theories and mathematical models are developed in both countries to describe hydrologic processes and impacts of watershed conditions on surface runoff. In the United States, however, theoretical investigations are directly pointed at wetlands and are supported by large-scale field studies and advanced technological capabilities to manage spatially distributed information. Unlike in Russia, in the United States, special scientific symposia are devoted to wetland hydrology, where major tasks for hydraulic and hydrologic research needs are formulated. Among these tasks are the understanding and assessment of relationships between various hydrologic modifications and wetland functions, especially wetland flood conveyance and water quality protection functions (1). Watershed-scale comprehensive field studies of wetland functions are underway, for example, at constructed experimental wetlands in the Des Plaines River basin in Illinois (2). A new long-term goal—strategic restoration of wetlands and associated natural systems—has been formulated (3).

The intensive efforts of many U.S. scientists yielded numerous results and attracted more attention to the complicated nature of wetlands processes. Wetlands were evaluated as runoff retention basins, and it was found that, in northwestern states, up to 12 inches of water could be accumulated per wetland acre (4, 5). Over time, piecemeal loss and degradation of wetlands in many areas of the United States have seriously depleted wetland resources. Researchers also discovered that adverse impacts from wetland degradation could appear indirectly with little obvious spatial or temporal connections to sources. As described by Johnston (6):

> Cumulative impacts, the incremental effect of an impact added to other past, present, and reasonably foreseeable future impacts, has been an area of increasing concern. . . . Impacts can accumulate over time or over space and be direct or indirect. An indirect impact occurs at a location remote from the wetland it affects, such as the discharge of pollutants into a river at a point upstream of a wetland system.

The process of solving environmental problems related to wetlands is increasingly complex. Analyzing diversified data over increasingly broad areas becomes essential for making competent decisions.

Comparing wetland hydrologic functions in headwaters of the Mississippi River (United States) and the Volga River (Russia) could provide additional information about how alternative management strategies affect runoff, peak flow, and water quality under changing climates. A macro-scale "field experiment" in both of these naturally similar areas is already under way. Wetland conservation as opposed to drainage is now the prevailing policy in the upper Mississippi basin. In Russia, however, economic problems have prevented this type of policy from becoming a priority. Instead, peat mining, reservoir construction on lowlands, and drainage for farming and private gardening are common.

This project, which is being implemented at the Natural Resources Research Institute (NRRI), University of Minnesota, Duluth, has the following goals:

- Developing a multilayered hierarchical base of geographic information system (GIS) data for headwater watersheds of the Mississippi and Volga Rivers.
- Developing a comparative analysis of wetland impacts on the hydrology of the rivers.
- Studying the relationships between natural and human-induced factors on wetland functions under climate change and variable strategies of wetland conservation.

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- Defining criteria and thresholds for wetland system stability with regard to flood risk and water quality.
- Outlining recommendations for wetland management in the headwaters.

The methodology for comparative assessments involves statistical analysis, hydrologic models, GIS, and remote sensing. Representative watersheds will be studied in more detail, and procedures for scaling information from the local to the regional level will be developed.

Input Data

In recent years, U.S. governmental and state agencies, as well as a number of private companies, have expended considerable efforts to compile the available data in GIS format for multidisciplinary analysis of watershed problems. Among the major sources of information essential for studies of wetland hydrologic functions are:

- The National Wetlands Inventory, conducted by the U.S. Fish and Wildlife Service.
- Digital elevation maps (DEMs) developed by the U.S. Geological Survey (USGS).
- Major and minor watershed boundaries, outlined for Minnesota by the Minnesota Pollution Control Agency (MPCA).
- The water quality sampling network from the U.S. Environmental Protection Agency (EPA).
- The digital chart of the world (DCW), issued by the Environmental Science Research Institute (ESRI) in scale 1:1,000,000.

These and other sources, listed in Table 1, were used to compile the map illustrations for this paper.

Almost no similar data in GIS form could be found for the territory of the former USSR, however. Any specific data (e.g., detailed maps, hydrology records, water quality sampling data) are generally in paper files dispersed among many agencies and are hard to obtain. The forms of information storage and means of its analysis are out of date, and most maps exist in single or few copies in paper files. In Russia, the time lag grows between the dynamic changes in the environment and the traditional pattern and inertia of management structures.

The GIS situation in Russia developed some interesting paradoxes. During the first few decades of space programs, certain state agencies accumulated an outstanding bank of world image data. When economic hardships hurt the previously privileged space industry, numerous joint ventures with foreign companies were created to distribute images on the world market for hard currency. These data are hardly available for domestic uses, however. The current domestic price for image data is 20,000 to 25,000 rubles for a black-and-white picture of an area 60 by 60 kilometers, or 60,000 to 70,000 rubles for the same image on a computer disk. With the present level of funding for scientific research, the price is too high. Security regulations still restrict access to later data, showing land use changes.

Another paradox is scientists' attitude toward their data. Abandoned by the state, agencies and institutes are reluctant to share their specific data in multidisciplinary projects. Data files are now a commodity. Accomplishing an overlay and integration of special data coverages, which is essential to any watershed GIS study, is almost impossible.

The third paradox is the attitudes of local, regional, and central authorities toward GIS. Many authorities are still ignorant about the potential of this technology. Those who are knowledgeable prefer not to promote GIS for watershed-related tasks because it involves land use analysis. With the onset of land privatization, the best pieces of property (e.g., the waterfront lots adjacent to drinking water reservoirs in the Moscow region) are rapidly allocated to the most powerful landowners. Thus, limiting access to this kind of information is deemed safer.

Experts in Russia have not yet applied GIS to wetland hydrology studies because GIS is still a very new and mostly unfamiliar technology. This makes the current study unique both for its results and for its application of GIS methodology.

Closer review of data sources indicates that most of the input data for the project is available, though dispersed among many agencies. Table 1 is a preliminary list of data and data sources.

Project Design

The project addresses the following questions:

- How do the extent and positioning of wetlands in the headwaters of large rivers affect runoff and peak flow?
- What are the spatial relationships between wetland and other land uses regarding flood risk and water quality under variable climate conditions?
- What is the role of wetlands for diffuse pollution prevention and sediment deposition control under alternative management?
- What determines major criteria for wetland conservation in headwaters, ensuring environmentally sustainable development under multiobjective land and water resource uses?

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Table 1. Data Sources for a GIS Study of Wetlands in the Basins of the Mississippi (United States) and the Volga (Russia)

Data Level 1	United States	Russia
Base maps	DCW[a]	DCW[a]
Stream network	DCW[a]	DCW[a]
Urban and rural areas	DCW[a]	DCW[a]
Wetlands, unclassified	NWI[a]	DCW[a]
Forests	DCW[a]	MGU[b]
Agricultural lands	LMD[b,c]	MGU[b]
Level 2		
Watershed boundaries	MPCA[a]	RWRC[c]
Digital elevation maps	USGS[a]	NA
Digital orthophotos	USGS, LMIC[a]	CD, RPI[b,c]
Soils	SCS[b,c]	RPI[c]
Hydrologic records	USGS[b]	CHM[b,c]
Water quality records	EPA/MPCA[a,b]	CHM, RCP, RPI[c]
Land uses	LMIC, LSAT[a]	LSAT, CD, RPI[a-c]
Wetlands, classified	NWI[a]	RPI[c]

[a]Data available in GIS ARC/INFO format.
[b]Databases; needed conversion to ARC/INFO.
[c]Data available in paper files; needed digitizing.

Key:

CD = Commercial distributors
CHM = Russian Committee on Hydrometeorology
DCW = ESRI digital chart of the world
EPA = U.S. Environmental Protection Agency
LSAT = Satellite image data
LMD = Published literature and map data
LMIC = Minnesota Land Management Information Center
MGU = Moscow State University
MPCA = Minnesota Pollution Control Agency
NA = Data not available
NWI = U.S. National Wetland Inventory
RCP = Russian State Committee on Natural Resources and Conservation
RPI = Miscellaneous planning agencies and research institutes
RWRC = Russian Water Resources Committee
SCS = U.S. Soil Conservation Service
USGS = U.S. Geological Survey

Tasks established to address these questions include:

- Developing a multilayered hierarchical base of GIS data for headwater watersheds of the Mississippi and Volga basins.
- Performing a comparative analysis and simulation of wetland impacts on hydrology and water quality at representative watersheds.
- Deriving the relationships between natural and human-induced factors and wetland functions under climate change with regard to variable strategies of wetland conservation.
- Defining the criteria and thresholds for wetland system stability with regard to flood risk and water quality parameters.
- Outlining recommendations for land and water resources management and wetland positioning in the headwaters.

GIS is the essential tool for manipulating and integrating the many types of spatial data on water resources, soils, vegetation, land use, economics, and the environment. GIS compiles many sources (e.g., maps, field notes, remote sensing, statistical data) into a consistent, interpretable database used for specific scientific goals and development decisions. The user can run GIS ARC/INFO software on workstation and PC platforms and apply the hierarchical approach to GIS data management, developed earlier (7). At the task level, data resolution and corresponding modeling tools vary.

Level 1 contains the basic reference information for large regions (e.g., the Upper Volga and the Minnesota portion of the Upper Mississippi River basins). It covers an area of several hundred thousand square kilometers, with a map scale approaching 1:1,000,000. Landsat thematic map data and the DCW (8) are used as sources of data at this level. Vogelmann et al. (9) demonstrated the methodology for detection of freshwater wetlands using remote sensing data based on maximum

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likelihood supervised classification. A graphic data file on GIS focuses on basic physical characteristics such as stream network, geology, soils, wetland classification, and other major land uses. A complementary tabular database or attribute file contains information on stream flow, water quality, pollution sources, and wetland impacts on material fluxes. At this level, the general physiographic and statistical information is accumulated and analyzed, territories are classified, and major problems and typical case study watersheds are defined. This information is compiled from literature, cartographic data in paper and digitized form, statistics, and space image data.

In Level 2, the more detailed GIS analysis and scenario-based modeling is implemented at the watershed scale with a map scale of approximately 1:25,000. The watershed demonstration focuses on alternative approaches to priority-setting in wetland management, climate impact analysis, and resulting interactions with landform, soils, biosphere, and runoff. The sources of data are special, topographic maps and air photo interpretation.

Simulation studies of water balance and fluxes among the various reservoirs are implemented at Level 2. Developing procedures for scaling information from the local to regional level is the important task at this level. GIS assists in handling the input parameter library and analyzing the output. GIS studies, involving area measurements and distribution analysis, evaluate cumulative impacts on runoff and its quality from the loss of wetland area, caused by drainage or filling, under stationary and changing climate.

Wetland functions are considered under two sets of scenarios. Management scenarios compare different wetland and farming allocations, conservation practices, and agricultural chemical use. Climate scenarios assume rainfall and temperature changes under global warming. Scenario-based simulation is applied in the analysis of watershed runoff, wetland moisture regime, soil erosion, and water quality processes.

Methodology

GIS database structure is related to the selected methodology. GIS serves as a linking tool for input-output data analysis and transfer between models, used at different levels and stages of studies.

Scientists in both the United States and Russia developed statistical methods to obtain quantitative relationships between stream flow and wetland area in the river basins. Johnston (6) summarized the U.S. findings:

> Empirical equations for predicting streamflow, developed by U.S. Geological Survey in Wisconsin and Minnesota, indicate that flood flow is proportional to the negative exponent of wetlands and lakes ratio on a watershed (10, 11). This means that relative flood flow is decreased greatly by having some wetlands in a watershed, but a watershed with a large proportion of wetlands does not reduce flood flow much more than a watershed with an intermediate proportion of wetlands. For example, predicted flood flow was 50 percent lower in Wisconsin watersheds with 5 percent lakes or wetlands than it was in watersheds with no lakes or wetlands, but increasing the proportion of lakes and wetlands to 40 percent decreased relative flood flow by only an additional 30 percent (12).

Other estimates agree that wetland encroachment on a watershed of less than 25 percent generally has a minimum influence on peak flow (5, 13, 14).

Johnston and colleagues (15) applied these equations to watersheds in central Minnesota. They found that a watershed with 1.6 percent lakes and wetlands had a flow per unit watershed area that was 10 times the flow predicted for a watershed with 10 percent lakes and wetlands, while watersheds with 10 to 50 percent lakes and wetlands had about the same flood flow per unit area.

Statistical analysis indicates that peak discharge increases with decreasing wetland area within the drainage basin. The regression equation defines the approximation for northwestern Minnesota (16):

$$Q_{AM} = 58.4\ A_W^{0.677}\ (L_S)^{-0.506}$$

$$L_S = 100(A_L + A_M)/A_W + 1$$

where:

Q_{AM} = arithmetic mean of the annual series, cubic feet per second.

A_W = watershed area, square miles.

A_L = lake area within the watershed, square miles.

A_M = marsh area within the watershed, square miles.

A similar statistical approach was developed for peak flow determination in Russia. Maximum flow discharge from snow melt is calculated for the central European zone as (17):

$$Q_m = K_0 * h_p * S_1 * S_2 * S_3 / (A + 1)^n$$

where:

Q_m = flow discharge, cubic meters per second.

K_0 = coefficient of flood concurrence, K0 = 0.006 for plain river basins.

h_p = calculated flood runoff for given probability, millimeters.

A = drainage area, square kilometers.

n = coefficient, n = 0.17.

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S_1 = lake storage coefficient, if lake area is less than 1 percent of A, then S1 = 1.

S_2 = pond and reservoir storage coefficient, S_2 = 0.9 with ponds and S_2 = 1 without ponds.

S_3 = combined wetland and forest storage coefficient.

S_3 = 1 - 0.8 lg (0.05 Sf + 0.1 S_w + 1), where S_f and S_w are forest and wetland area, percentage to total drainage area.

Calculated flood runoff for a given probability, h_p, is determined based on average flood runoff h, millimeters, coefficient of variation C_v, and tabulated parameter F, as follows:

$$h_p = (1 + F * C_v)\, h$$

$$h = K_t\, h_k$$

h_k = 100 millimeters for Moscow region.

K_t = land surface coefficient, Kt = 0.9 for plains and sandy soils, K_t = 1.1 for hills and clay soils.

The studies mentioned above generally agree with an assumption that the incremental loss of wetland area would have a small effect on flood flow from watersheds with 10 percent up to 40 to 50 percent wetlands, but a large effect on flood flow from watersheds with less than 10 percent wetlands.

The existence of similar thresholds was found in relation to wetlands abilities to intercept pollutants. As Johnston stated (6),

> The same 10 percent threshold was identified by Oberts (18) for suspended solids, a measure of water quality function. Stream-water draining watersheds having 10 to 20 percent wetlands had about the same loading of suspended solids, so the contribution of suspended solids was relatively constant per unit area of watershed. However, the watersheds with less than 10 percent wetlands had loading rates per unit area that were as much as 100 times greater than the loading rates from the watersheds with more than 10 percent wetlands.

GIS could be especially helpful in determining the impacts on downstream water quality of the spatial positioning of wetlands within watersheds. Studies prove that the location of wetlands can affect their cumulative function with regard to water quality (6). In an earlier work (15), Johnston developed an index of wetland location and applied it to a landscape-level GIS study of urban and rural stream watersheds in central Minnesota. The index is formulated as:

$$PWP = \sum_{i=1}^{j} A_i / \sum_{i=1}^{j} A_i$$

where:

PWP = relative wetland position.

j = stream order (19) of water quality sampling station.

i = stream order of wetland.

Ai = area of ith order wetlands.

Calculated values for the index ranged from 0 (i.e., all wetlands were on streams of the same order as that of the sampling station) to 2.6 (i.e., average wetland position was 2.6 stream orders upstream of the sampling station). Watersheds with wetlands located close to sampling stations had significantly better water quality (i.e., lower concentrations of inorganic suspended solids, fecal coliform, and nitrate; lower flow weighted concentrations of ammonium and total phosphorus) than watersheds with wetlands located far from sampling stations (6).

The review of methodological approaches, as shown above, indicates that parameters describing wetland extent, positioning, and land surface characteristics are of universal significance for any comprehensive watershed-scale wetland study.

In the current study, GIS is used at Level 1 to evaluate wetland area per watershed and to develop input parameters for relative wetland position assessment. The comparison and selection procedures for case study watersheds in the Volga and Mississippi basins are based on these values. The parameters, derived from GIS, are as follows:

1. Total watershed area.
2. Lake, pond, and reservoir area.
3. Forest area.
4. Wetland area.
5. Ratio of wetland area to total watershed area.
6. Wetland area by subwatersheds of different order.
7. Relative wetland extent by subwatersheds of different order.
8. Land surface coefficients.

Parameters listed in groups 1 through 4 are obtained directly from GIS attribute tables as values of "area" items for the respective land cover polygons. Parameters 5 through 7 require calculations relating values of area items for different polygon coverages. Land surface coefficients (group 8) could be determined indirectly based on basic soil, land cover, and topography data. Most U.S. methodologies use hydrologic soil groups,

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based on soil permeability, rates of infiltration, and Soil Conservation Service (SCS) runoff curve numbers (20). Some Russian methodologies have adopted similar empirical land surface coefficients. For example, for central European Russia, this value varies from $K_t = 0.9$ for plains and sandy soils, to $K_t = 1.1$ for hills and clay soils (17).

Level 2 of analysis applies two hydrologic simulation models:

- The Agricultural Watershed Runoff and Water Quality Model (Agricultural Nonpoint Source Pollution Model [AGNPS]), developed by the Agricultural Research Service of the U.S. Department of Agriculture, contains explicit procedures to evaluate the impacts of management practices and landscape feature positioning on watershed runoff. AGNPS is a cell-based runoff model that estimates water volume, peak flow, eroded and delivered sediment, chemical oxygen demand, and nutrient export from watersheds (20-22).
- The Forest Runoff Watershed Model (FRWM) combines analytical and numerical methods for solving hydro- and thermodynamics equations (23). This model considers snow melt constituent in runoff in more detail than does AGNPS. Hydrologic simulation is based on physical process descriptions for snow cover dynamics, freezing and thawing of soil, soil moisture dynamics in frozen and thawed soils, interception of liquid and solid precipitations by vegetation, surface runoff, ground-water aquifers, and channeled streams.

Both models use a similar set of watershed input data, derived from GIS (e.g., elevations, slopes, channel slopes, stream network configuration, soil texture, land cover). The methodology, linking GIS with hydrologic models, was already tested in the wetland study project at the Voyageurs National Park in Minnesota. The ARC/INFO GRID module was used to derive watershed variables for input to AGNPS. GIS then presented and interpreted the scenario-based results of the simulation (24). The typical stages of such an analysis and interpretation for a watershed-scale area are presented in Figures 1 through 5.

Case Study Watersheds

The areas where wetland impacts on runoff are evaluated are located in Minnesota (United States) and Moscow and adjacent regions (Russia) (see Figures 6 through 15). They have mixed urban, rural, recreational, and forest land uses. Both regions have a variety of development pressures. The relative effects of different alterations in watershed management are distinguished and quantified. GIS provides metrics for comparative assessments and analysis of related variables for both areas.

Table 2 and Figures 7 through 14 present a general overview of wetland extent in both areas. The case study subwatersheds used for more detailed analysis will include tributaries of the second and third order. At this stage, several watersheds are considered for more detailed analysis. The limitations imposed by data availability as well as by project resources could affect the final selection. Table 2 serves, therefore, as a preliminary overview of several areas that could potentially be adopted for more detailed studies.

GIS analysis shows that in the Upper Volga, wetlands extent very much depends on allocation of populated areas. The heavily urbanized Moscow metropolitan area affects a large territory of many thousands of square kilometers. The ratio of wetlands as a percentage of total land area is one-tenth of that in the neighboring Tver area, which has the same size but a smaller population (see Figure 11). In areas of intensive agriculture (e.g., the Pronya basin located southeast of Moscow), almost all wetlands were drained and have not existed for several decades.

In Minnesota, despite the growing urbanization (e.g., the Twin Cities area [7,330 square kilometers]), about half of the presettlement wetlands still remain (25); wetlands occupy 442 square kilometers, or over 6 percent of the land area; and shallow lakes constitute an additional 114 square kilometers (1.56 percent). Some watersheds within the Twin Cities metropolitan area have a high wetland percentage, such as 18.9 percent in the Lamberts Creek watershed. Intensive studies with GIS application of landscape feature functioning and wetland impacts on stream flow and water quality demonstrated an innovative approach and made detailed databases available for this area (15).

Preliminary comparative analysis indicates that two pairs of case study watersheds could be initially selected for further studies in the Mississippi and Volga basins:

- Upstream watersheds with wetlands area of 15 to 20 percent (Tver region in Russia and Cass and adjacent counties in Minnesota).
- Tributary watersheds downstream with wetlands area of 1 to 2 percent (the Istra basin in Russia and subwatersheds of the Minnesota River basin, located in Sibley, Scott, and adjacent counties in Minnesota).

Case study watersheds in the Mississippi and Volga basins are situated on gently rolling plains in mixed forest zones with southern portions extending into the forest/steppe and prairies. The Quaternary sediments are of glacial, glaciofluvial, lacustrine, and alluvial origin. Wetlands have hydric soils with various degrees of gley process development and/or peat accumulation, varied by wetland type and soil moisture regimen (28). The annual precipitation is 500 to 600 millimeters with similar

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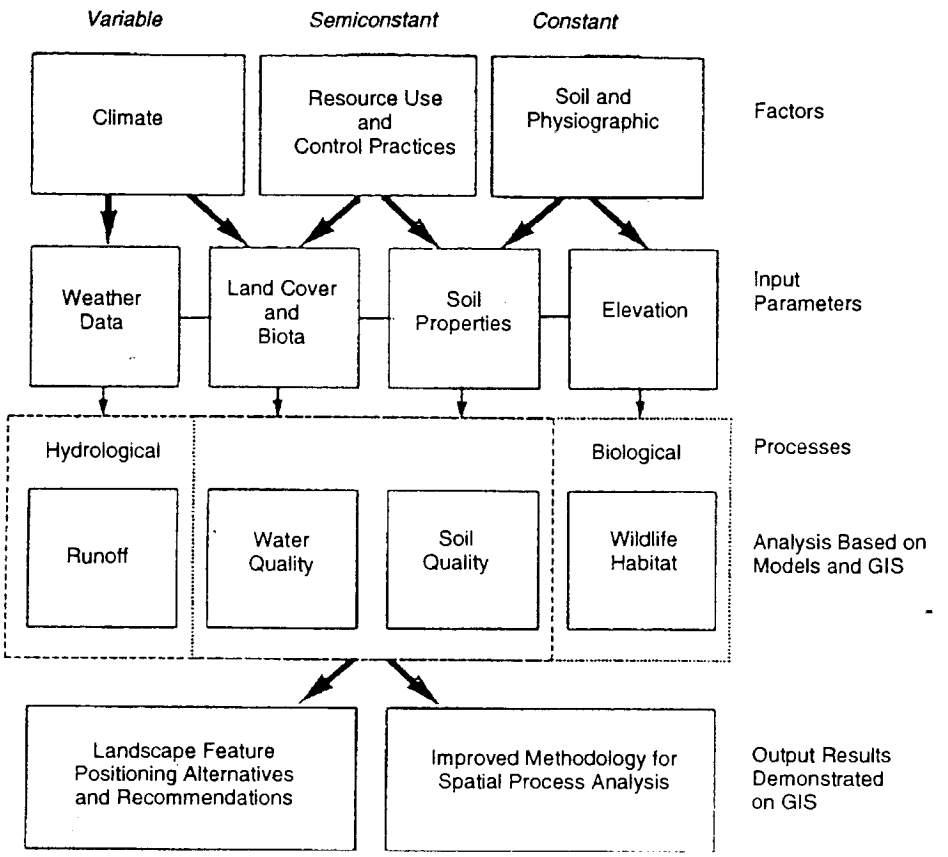

Figure 1. Conceptual framework of linking GIS and models for environmental management.

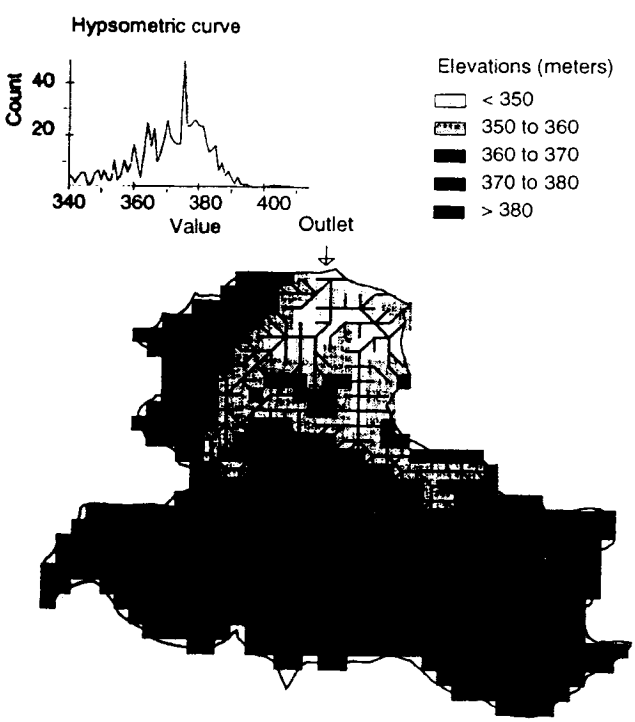

Figure 2. Stream network configuration derived from GIS elevation map.

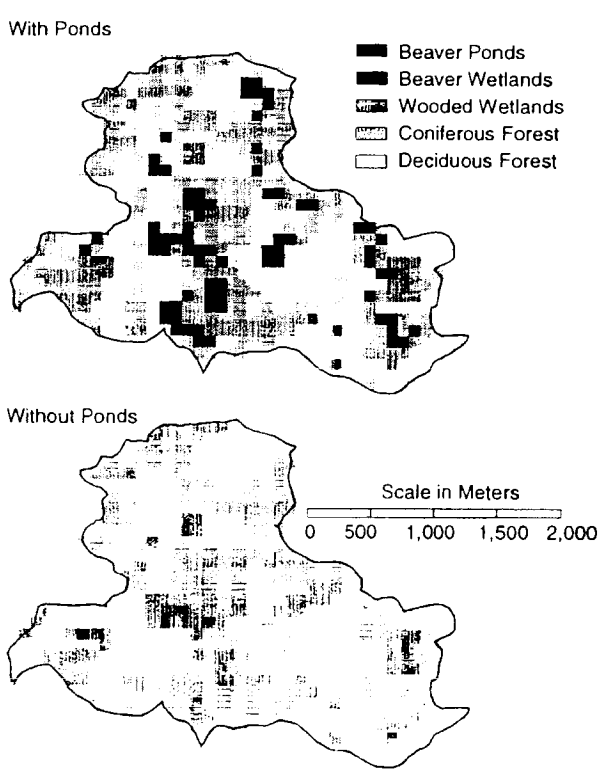

Figure 3. Scenarios of land use.

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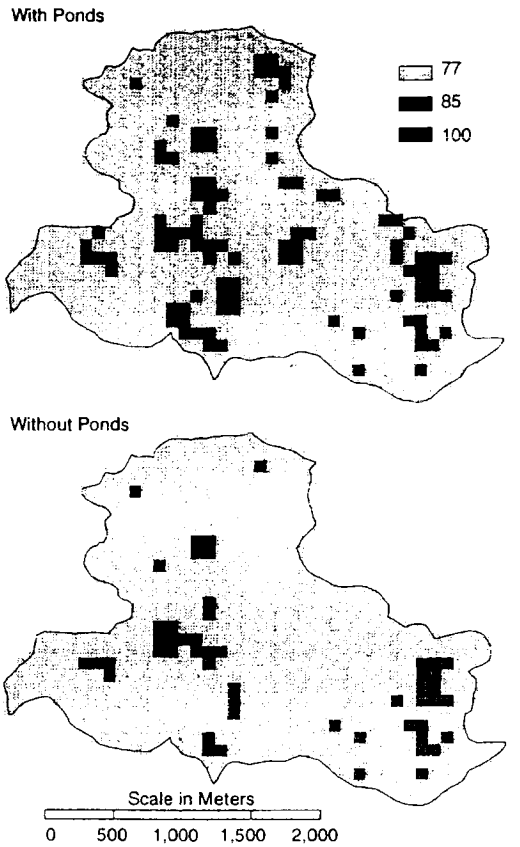

Figure 4. Scenario-related hydrologic curve numbers.

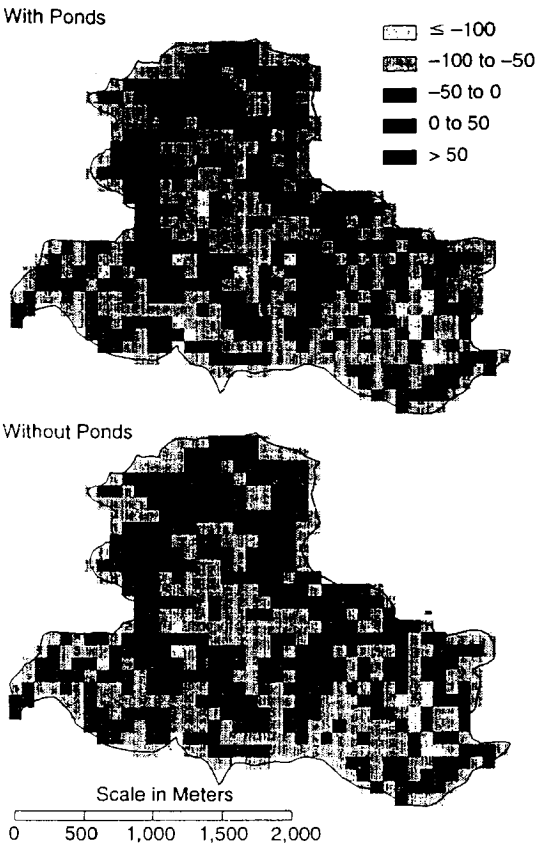

Figure 5. Patterns of sediment transfer between cells (%), + deposition, − erosion.

Table 2. Comparative Data on Wetland Extent in Minnesota and in the Upper Volga Basin (8, 25-27)

Region	Total Area (Square Kilometers)	Wetland Area (Square Kilometers)	Percentage
United States			
Minnesota	205,940.30	30,500.00	14.80
Beltrami Co.	7,923.04	3,909.23	49.34
Cass Co.	6,256.38	1,505.29	24.06
Hubbard Co.	2,624.81	283.22	10.79
Le Sueur Co.	1,204.82	28.31	2.35
Hennepin Co.	1,588.07	36.37	2.29
Sibley Co.	1,555.38	24.26	1.56
Wright Co.	1,852.97	24.27	1.31
Scott Co.	982.56	8.06	0.82
Lambert Creek	9.51	3.69	18.90
Russia			
Tver region	10,000.00	1,169.08	16.90
Moscow region	10,000.00	165.39	1.70
Istra basin	1,827.38	24.07	1.32
Pronya basin	10,200.00	[a]	[a]

[a]Wetland area is insignificant and not identified by available maps.

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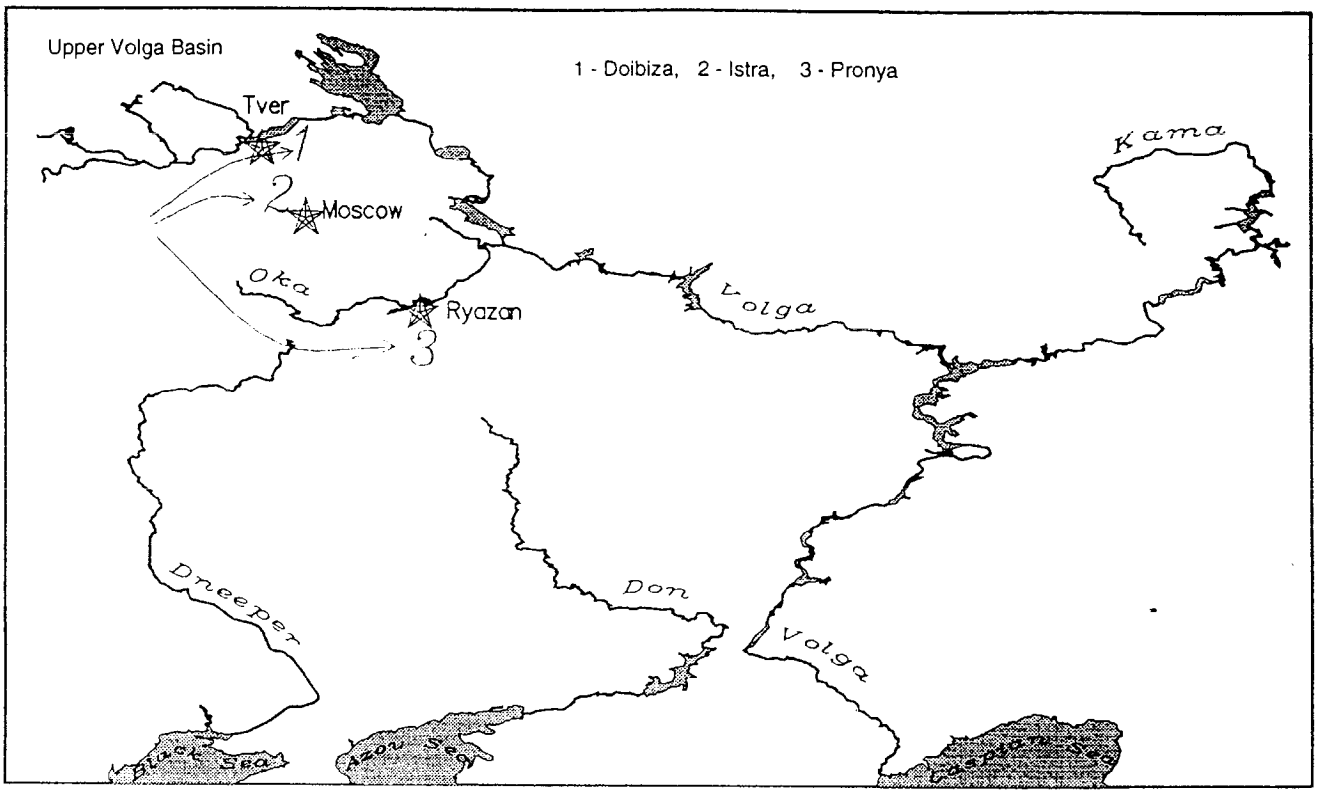

Figure 6. Location of study areas in the Volga basin.

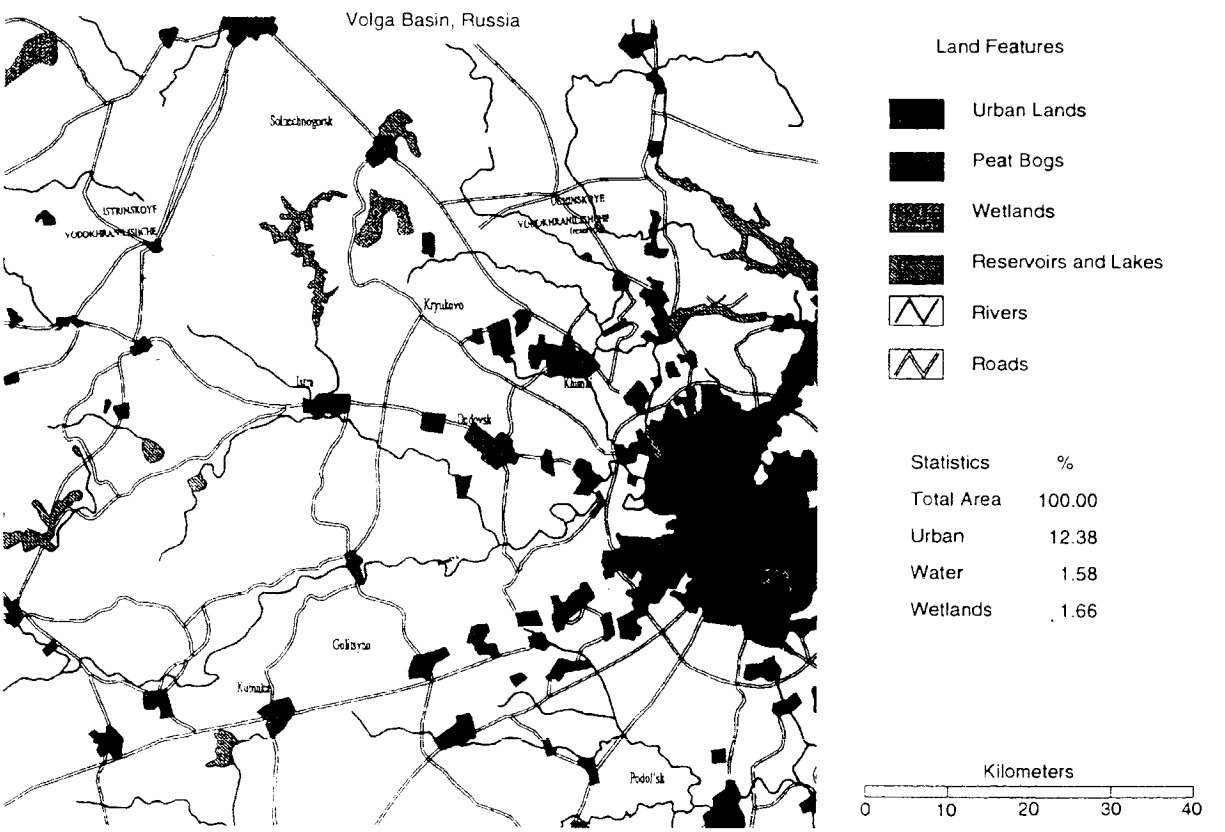

Figure 7. Wetlands and urban lands in the Moscow region.

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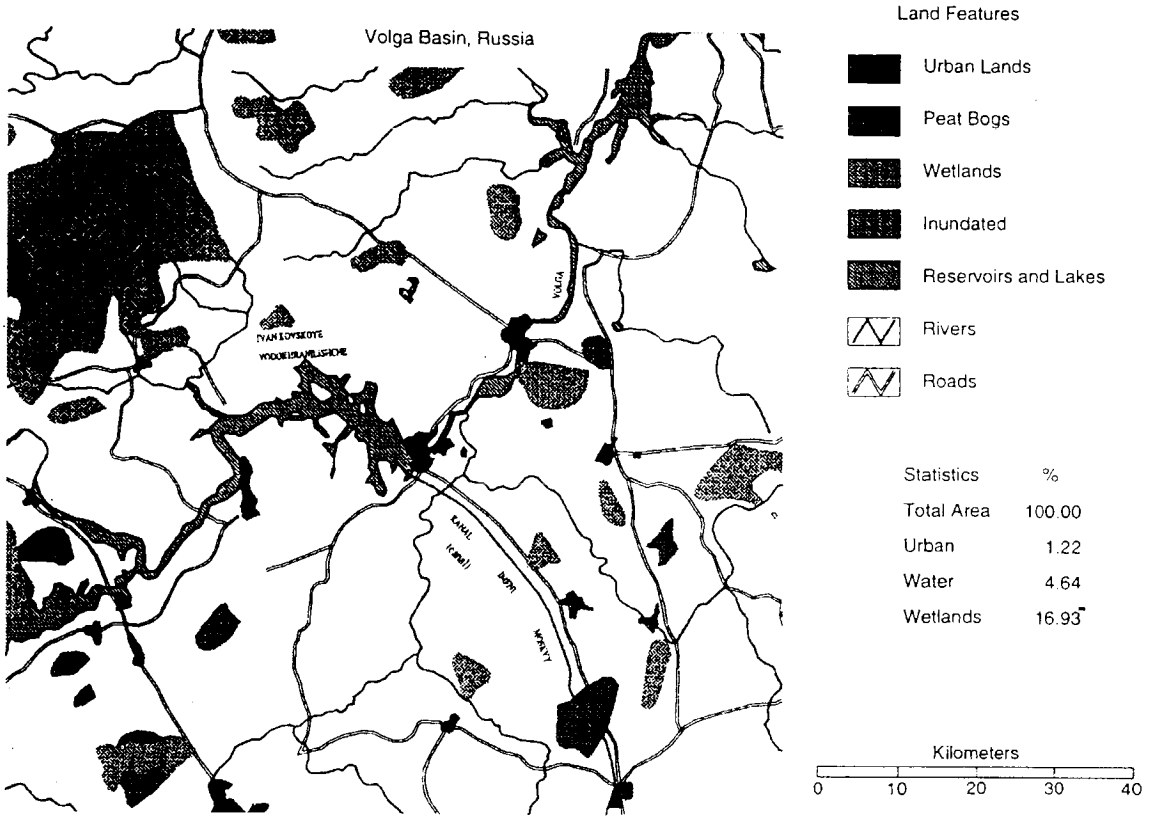

Figure 8. Wetlands and urban lands in the Tver region.

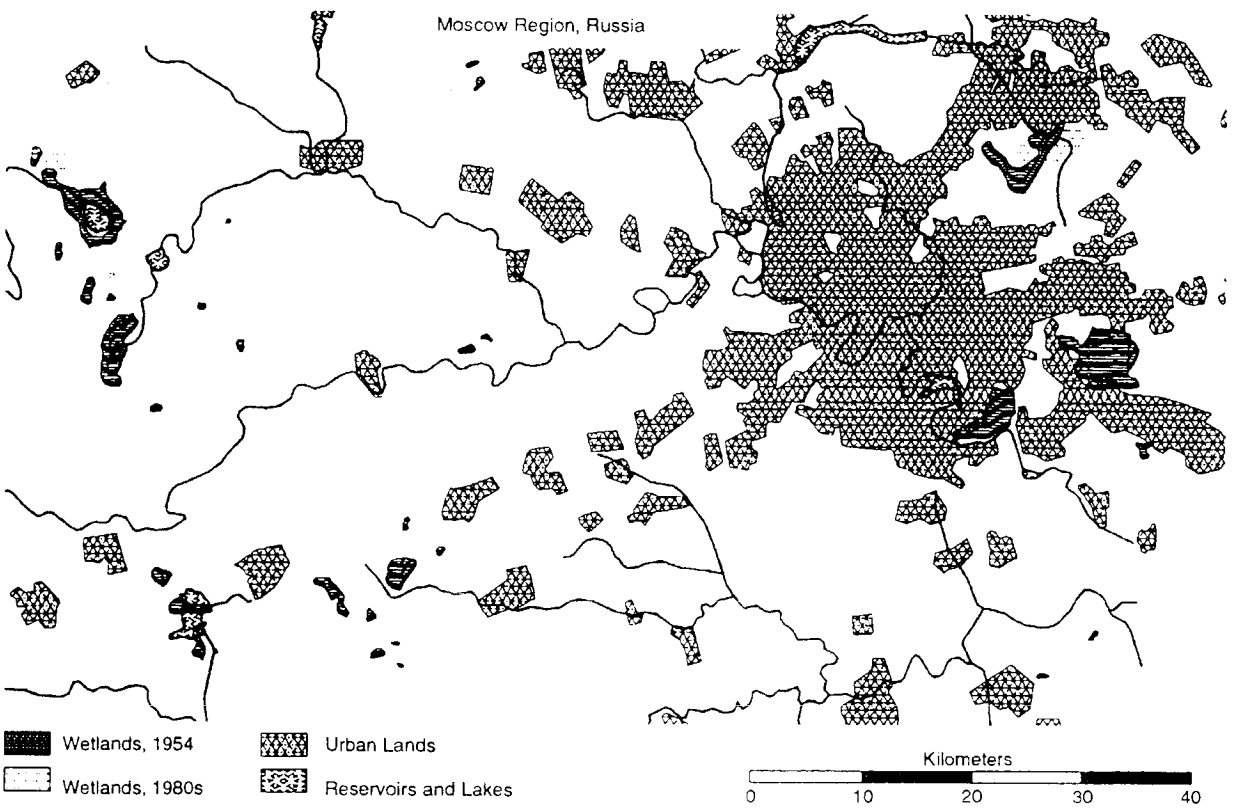

Figure 9. Wetland decline since 1954, Moscow region.

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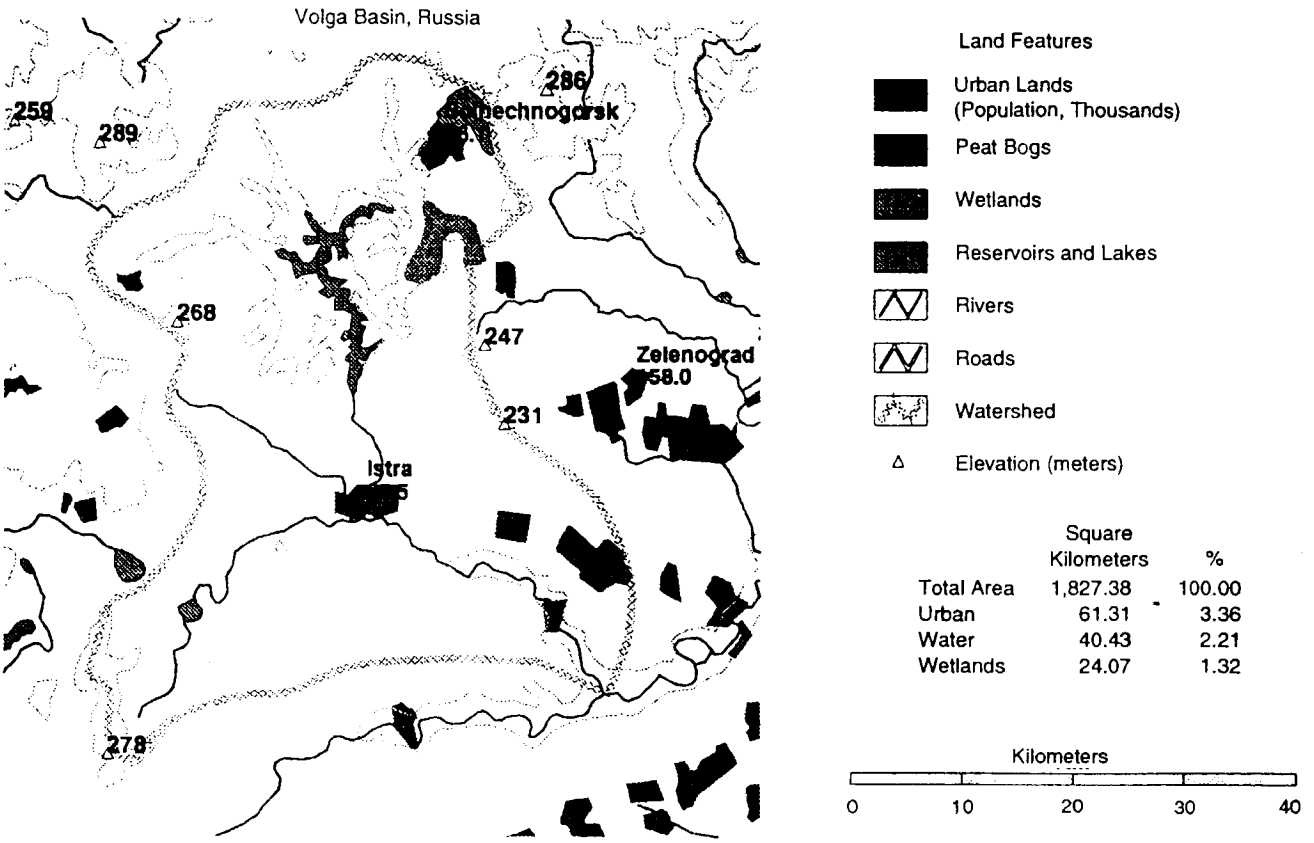

Figure 10. Istra watershed in the Moscow region.

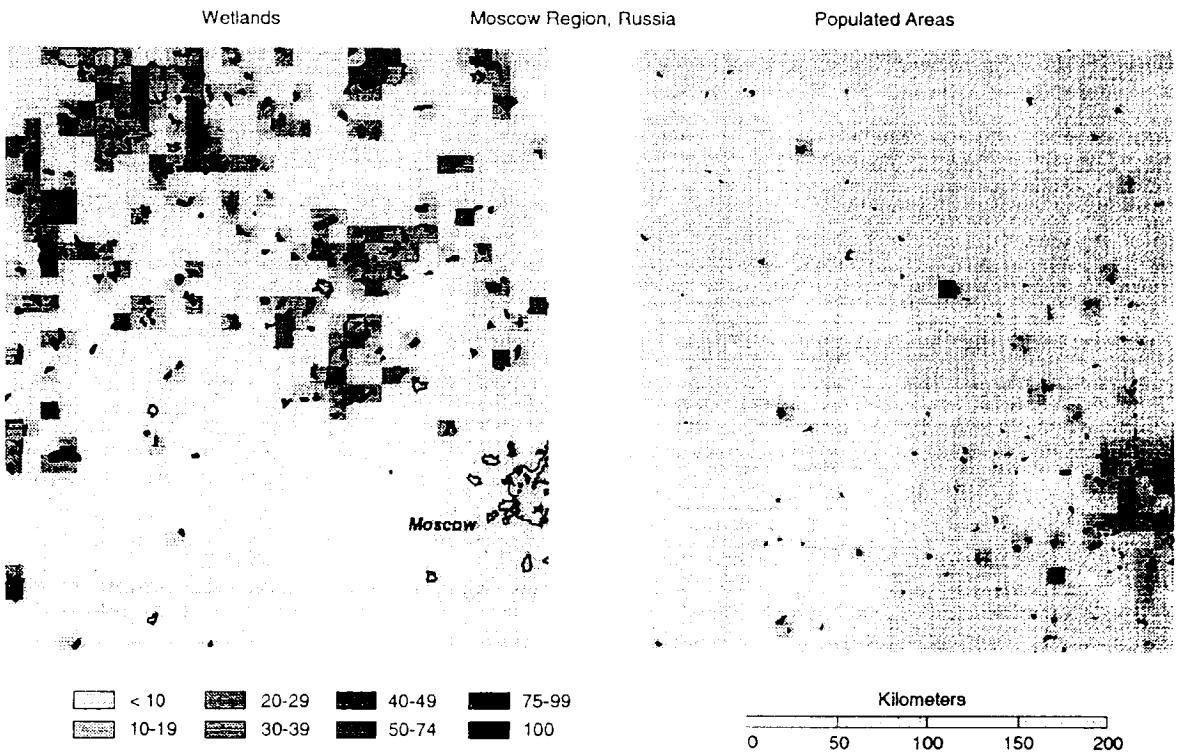

Figure 11. Land cover, percentage of total in Moscow and Tver regions.

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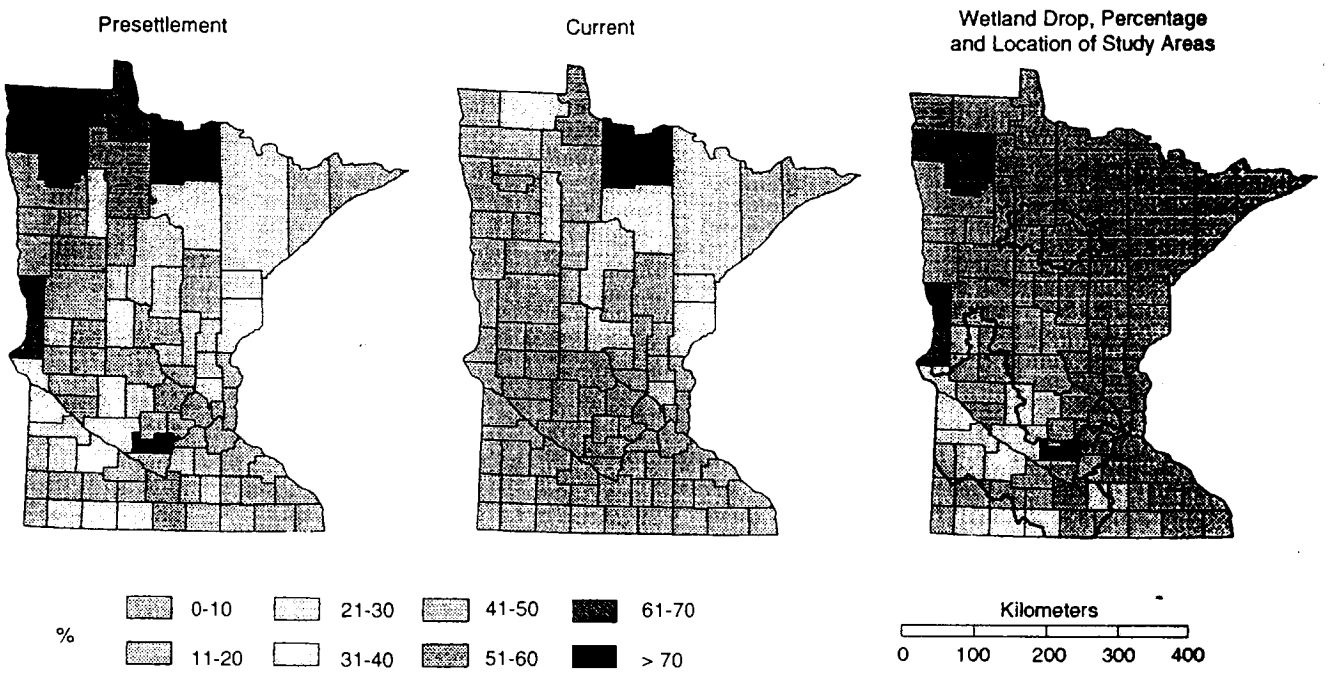

Figure 12. Wetlands in Minnesota, percentage of total area.

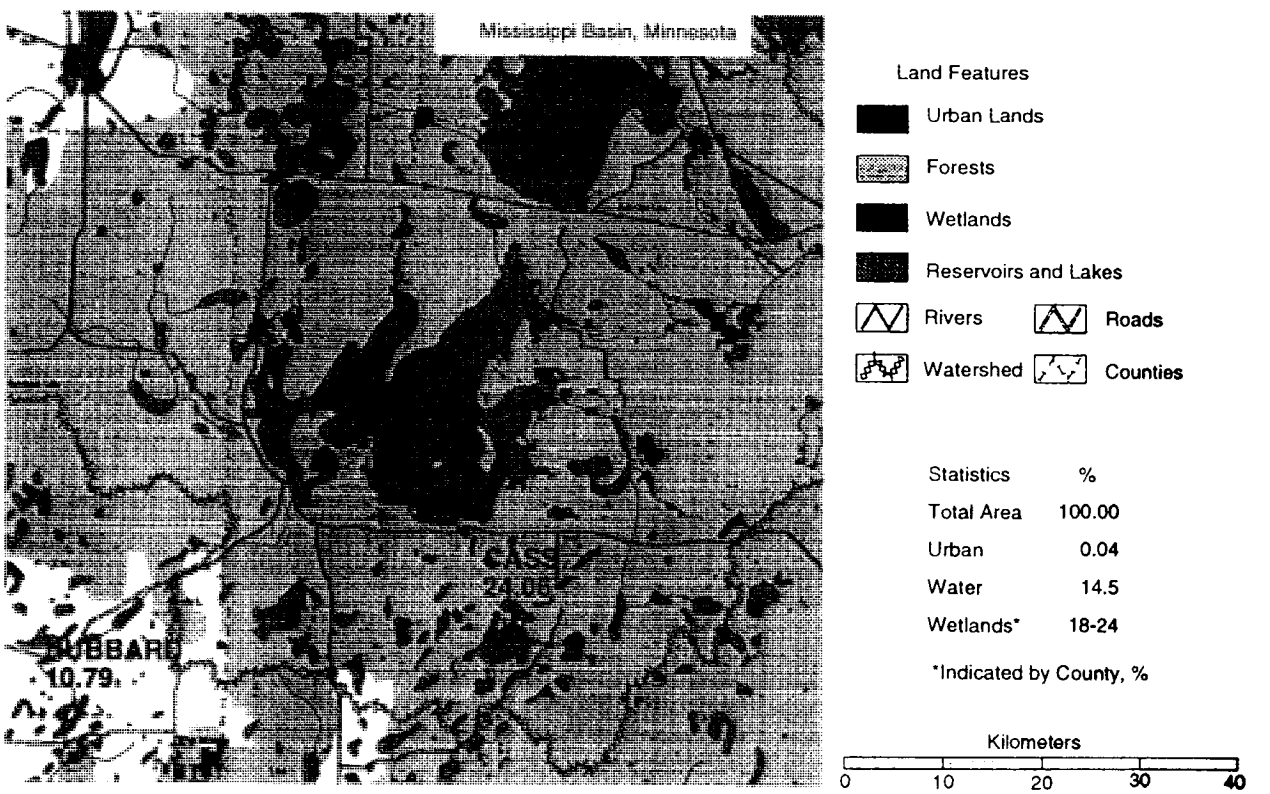

Figure 13. Wetlands and urban lands in Cass County area.

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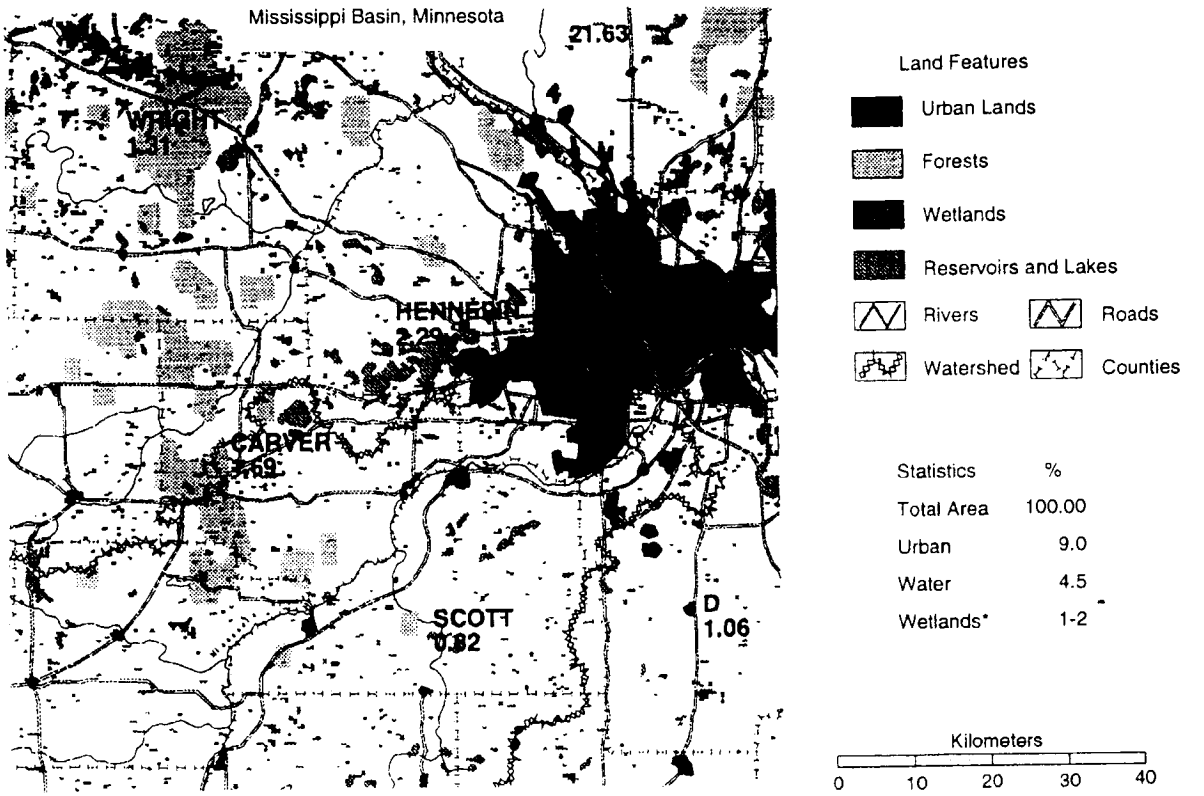

Figure 14. Wetlands and urban lands in Twin Cities area.

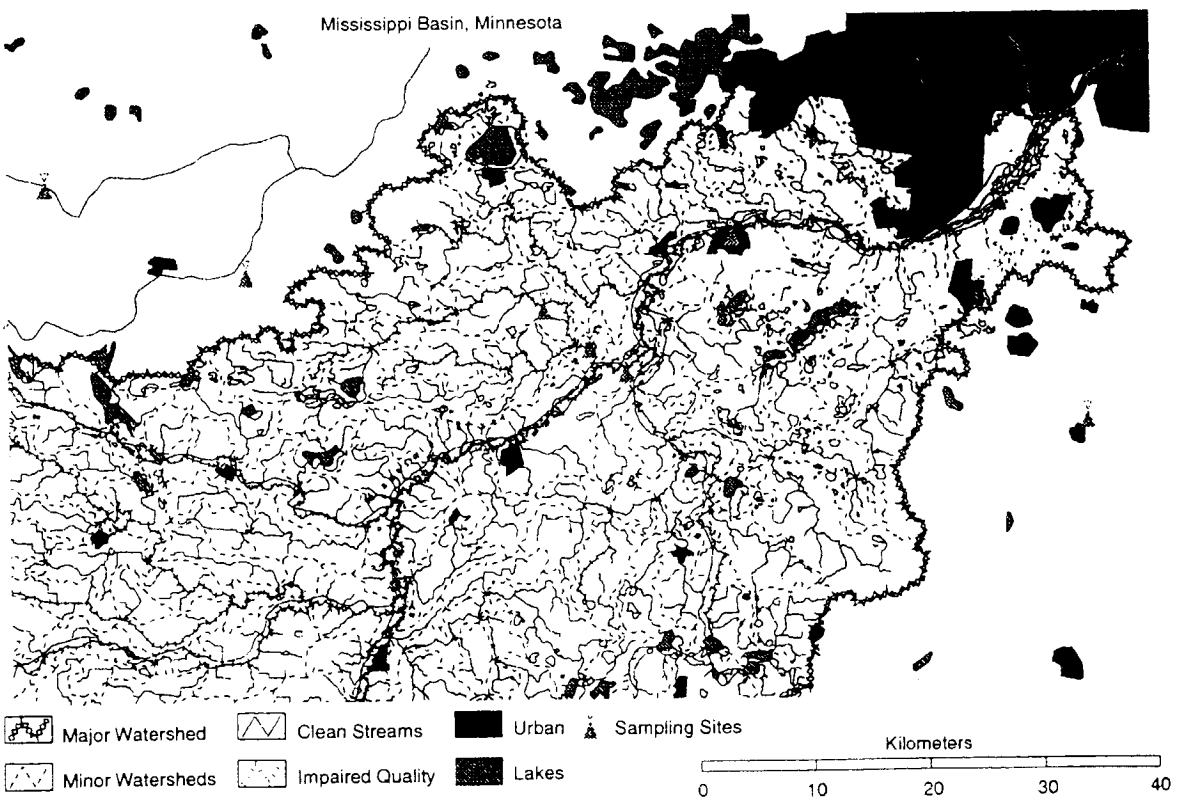

Figure 15. Minnesota River watershed in Twin Cities area.

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seasonal distribution in both areas. Average runoff ranges from 150 to 250 millimeters (7, 29).

Conclusion

The current status of the project indicates that most of the input data is available, though dispersed among many agencies. In both the United States and Russia, research methodologies have been developed and applied to study landscape feature impacts on runoff quantity and quality based on simulation and statistical analysis. The comparative analysis of hydrologic and diffuse pollution processes on watersheds in the Upper Mississippi and Upper Volga basins will allow derivation of metrics of wetland loss relative to impacts on runoff and water quality.

The applications of GIS to watershed hydrology are currently much more advanced in the United States than in Russia. Initiatives emerging in the United States, however, could considerably promote GIS use in Russia. Such promotion is beneficial for several reasons. First, this kind of cooperative political activity is in full compliance with the 1992 Freedom Support Act, approved by the U.S. Congress. Second, support of GIS as a new information technology will create a favorable infrastructure in many bilateral economic fields and businesses. Third, a better meshing of the GIS systems in the two countries will lead to further international cooperation in responding to global changes.

Project implementation also helps meet the goal of providing a basis for sound environmental, technical, and economic decision-making on the use of natural resources. This knowledge is essential in developing practical guidelines for sustainable economic development through applied research and technologies.

Acknowledgments

The Water Quality Division of MPCA provided valuable assistance with GIS data for Minnesota. The National Science Foundation (NSF/EAR-9404701) contributed research support. Any opinions, findings, and conclusions or recommendations expressed in this material are those of the author and do not necessarily reflect the views of the National Science Foundation.

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Water Quality Applications

Vulnerability Assessment of Missouri Drinking Water to Chemical Contamination

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Introduction

In 1991, the Missouri Department of Natural Resources (MDNR) implemented the Vulnerability Assessment of Missouri Drinking Water to Chemical Contamination project. MDNR's Public Drinking Water Program (PDWP) contracted with the Center for Agricultural, Resource, and Environmental Systems (CARES) to conduct this assessment. They designed the project to determine which, if any, public water supplies are threatened by chemicals being tested under the Safe Drinking Water Act.

Under Phase II of the Safe Drinking Water Act, the United States Environmental Protection Agency (EPA) required that all public drinking water systems be routinely monitored for 79 contaminants beginning January 1, 1993. If a selected chemical parameter is not detected in an area that would affect a water supply (where "detected" is defined as used, stored, manufactured, disposed of, or transported regardless of amount), then the water supply need not be tested for that chemical. Instead, that system would be granted a use waiver, meaning that the state would not test for that chemical. EPA grants use waivers for 43 of the 79 contaminants. Use waivers can result in considerable cost savings.

Because use waivers are granted based on the spatial relationship between drinking water sources and contaminant sources, accurate positional data needed to be collected for those items. A geographic information system (GIS) was used to store and analyze this information in a spatial context.

Water Sources

Water sources, as defined for this study, are the points where water is drawn from a river, lake, or aquifer for use in a public water supply. Our efforts focused primarily on the development of the water source layers for the GIS. These layers, containing wellheads, impoundment intakes, and river intakes, were created in house or obtained from state and federal agencies. MDNR regional office personnel inspected these water source layers in the spring of 1993. Since these personnel routinely inspect Missouri public drinking-water supplies, their knowledge of these locations is exceptional. The updated water source information was mapped on 1:24,000-scale USGS topographic quadrangles at the regional offices, then entered into the GIS. MDNR's PDWP provided available attribute information, which was associated with these layers. The layers offer the most accurate and current information available. Only the community (e.g., cities, subdivisions, mobile home parks) and the nontransient, noncommunity (e.g., schools, large businesses) water supply systems were considered for water source mapping. This study did not consider private wells.

The information is stored in the GIS in the form of geographic data sets or layers. The wellhead layer contains 2,327 public wells and their attributes (e.g., well depth, casing type). The majority of the wellheads are located in the Ozarks and Southeast Lowlands. Naturally poor ground-water quality prohibits a heavy reliance on ground water for drinking water in other areas of the state. The surface water impoundment layer contains 105 points representing the intake locations for systems that rely on lake water. Additionally, the drainage basin and lake area are mapped for these systems. The majority of the systems that rely on lake water are located in northern and western Missouri. The final layer represents the systems that use river water. The majority of the 50 intakes are located on the Mississippi and Missouri Rivers and on the major streams in the Grand and Osage River basins.

Contaminant Sources

Contaminant sources, as defined for this study, are the points or areas where existing databases indicate the presence of a chemical contaminant. Incorporation of contaminant data into the GIS proved to be the most difficult task. These data usually contained very precise

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information about what contaminants were found at a site and who was responsible, but the quality of the locational information was often poor.

Ninety-three state and federal databases were reviewed for contaminant information before performing the final use waiver analysis. The contaminant information was broken into two separate types, contaminant sites and pesticide dealerships. The contaminant sites were locations at which certain chemicals were known to exist. The pesticide dealerships were dealerships licensed to distribute restricted use pesticides. Information about contaminant sites was extracted from the databases and entered into Microsoft Excel, a spreadsheet program. The small amount of data with coordinate (latitude/longitude) or map information was readily converted to the GIS. The majority of the contaminant records, however, contained only address information, often appearing as a rural route address or post office box number.

While the water source locations were being verified, personnel at the MDNR regional offices reviewed the contaminant site records. The regional office personnel were familiar with their respective territories and could assist CARES personnel in locating the contaminant sites. The Missouri Department of Agriculture pesticide use investigators provided additional information about the locations of contaminant sites. All contaminant source information was also mapped on the 1:24,000-scale USGS topographic quadrangles and transferred to the GIS.

Of more than 2,800 contaminant sites found in these databases, 88 percent were geographically located and used in the study. At this time, the contaminant site layer contains 2,493 points representing the information collected on the 43 chemical contaminants required by MDNR. Each point contains a seven-digit chemical code indicating the chemical it represents and serving as a link to the chemical contaminant files. The contaminant sites tend to be concentrated more in urban areas than rural areas. Even though this layer is being continually updated, the basic distribution of contaminant sites remains the same.

A second contaminant source layer represents Missouri's licensed pesticide dealers. This information is included to indicate potential contamination even though specific chemicals at dealership locations are not known. At this time, we have been able to locate 1,344 dealerships out of 1,650. Two types of dealerships are included in the layer, active dealers and inactive dealers. Of the active dealerships in 1991, 91 percent were found and entered into the GIS. Of the inactive dealerships, 79 percent were located.

Spatial Analysis

The final parameters for the use waiver analysis were developed from EPA and MDNR guidelines and account for the capabilities of the GIS. These parameters were designed to present a conservative list of the systems that needed to be tested for the possible presence of studied chemicals. Parameters for the wellhead analysis are as follows:

- A ¼-, ½-, and 1-mile radius around each wellhead was searched for contaminant sites and pesticide dealerships (see Figure 1). Any contaminant sources found within those radii were reported to PDWP. (PDWP requested that the results of the three radius analyses be reported, but the ½-mile radius was used to determine the issue of the use waiver.)
- Any wellheads found within a contaminant area were denied a use waiver for that contaminant.
- Each highway and railroad within 500 feet of a wellhead was recorded. This indicates the threat posed by the transport of chemicals near wellheads.
- Additionally, the percentage of the county planted in corn, soybeans, wheat, sorghum, tobacco, cotton, and rice was listed for each well to indicate the threat posed by agricultural chemical use within that county.

The parameters for the systems relying on lake water are as follows:

- Any contaminant sources found within a surface water impoundment drainage basin caused the associated intake(s) to fail use waiver analysis for those contaminants.

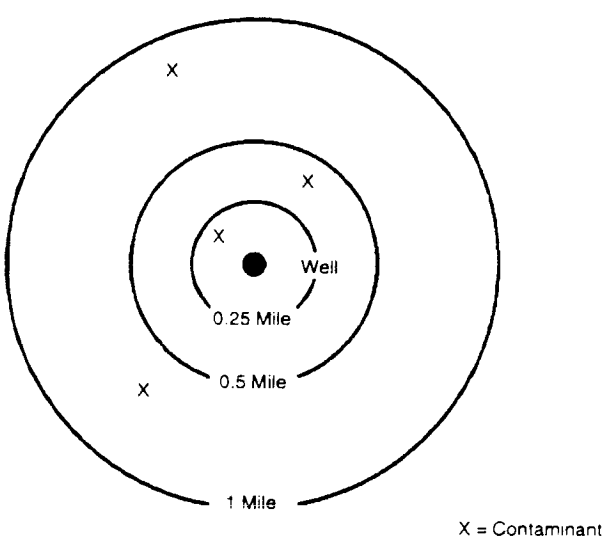

Figure 1. Use waiver search radius distances.

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- Any area of contamination overlapping a drainage basin caused the associated intake to fail use waiver analysis for that contaminant.
- Transportation corridors passing through a drainage basin were noted to indicate the threat posed by transport of chemicals within the basin.
- The percentage of the county planted in the seven crops mentioned above was listed to indicate agricultural chemical use within the drainage basin.

Many of the rivers that supply water to systems in Missouri have their headwaters outside the state. To fully evaluate the potential for contamination within those drainage basins, we would have to collect data for large areas outside of the state. For example, the Mississippi and Missouri River drainage basins cover large portions of the United States. Because collecting data for those areas would be impractical, we have recommended to MDNR that use waivers not be granted to river supplies.

The following provides details on how the analysis was performed. The GIS searches around each wellhead for each radius and notes which contaminant sites affect which wellheads. If a contaminant falls within that radius, we recommend that the wellhead be monitored. In this example, the well is affected by one contaminant within the ¼-mile radius, two within the ½-mile radius, and four within the 1-mile radius.

Results

The results of the use waiver analysis indicate which systems may be affected by the use of a chemical near a water source. Several results show the substantial savings realized from our analysis. For example, the analysis showed that only five wells serving four public drinking water systems were potentially affected by dioxin and should be monitored. By not testing the remaining systems for dioxin, the state can realize a considerable cost savings, as the test for dioxin is the most expensive test to perform.

The final wellhead system analysis shows that the ½-mile buffer analysis affected a total of 447 wellheads in 241 systems. That is, a chemical site or pesticide dealership was found within ½ mile of 447 public wellheads. A result form was generated for each of the 1,340 systems in the state listing each well or intake and the potential threat posed by nearby contaminant sources.

The cost of testing all wellhead systems for all 43 contaminants without issuing use waivers is more than $15 million (see Table 1). According to our analysis, CARES estimates that only $1.8 million need be spent to monitor vulnerable wells. Therefore, the state can save more than $13.5 million in monitoring costs.

Table 1. Estimated Cost Savings for Public Drinking Water Systems

Method	Estimated Total Cost	Estimated Mean Cost per System	Estimated Total Cost Savings
No use waiver	$15,533,100	$12,200	$0
With use waiver	$1,813,900	$1,400	$13,719,200

Summary and Recommendations

To date, the investment the state made in the vulnerability assessment project has provided many benefits. The state saved several million dollars in testing costs and developed several spatial and nonspatial databases that will have many uses. In addition, the project established a basic framework for future assessments, which EPA requires on a regular basis.

The basic data required for use waiver analysis are the locations of water sources and the locations of potential contamination sources. CARES determined that the available data did not contain the information necessary to map these locations or that the data were of questionable quality. Many layers required update and correction. Considerable effort was necessary to improve existing locational information for both water source layers and chemical contaminant files. Local knowledge of an area was heavily relied upon to determine accurate locations, particularly contaminant sites. The vast majority of these sites contained only the address as the geographic reference. An address is not a coordinate system; it does not indicate a fixed location on a map. Because the location of any chemical detection site is of vital importance, state and federal agencies that collect these data need to record more complete geographic information. Ideally, a global positioning system could be employed to generate coordinates. Realistically, the recording of legal descriptions or directions from an easily located point would substantially improve the quality of the current databases.

In many cases, data resided in digital format; however, due to regulations or lack of agency cooperation, they could only be distributed in paper format. Reentering data from paper format into digital format required considerable time and expense. Interagency cooperation should be emphasized to reduce unnecessary data entry.

R0021856

Reach File 3 Hydrologic Network and the Development of GIS Water Quality Tools

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Introduction

The application of geographic information system (GIS) tools to water quality management is limited by the lack of geographically referenced data describing the surface water environment. Ongoing efforts at the local, state, and federal level are producing a multitude of GIS data coverages describing land use/cover and relevant water quality data files. As these data coverages become available, water quality managers will need to develop new analysis techniques to take advantage of the vast amount of geographically referenced data. A key step in the development of analytical tools for water quality management will be the development and maintenance of a coverage describing the structure and hydrology of surface waters.

Reach File 3 (RF3) is one potential source of surface water maps and topology for the development of a GIS-based water quality analysis tool. This paper describes a pilot project designed to examine the suitability of RF3 as a network system for the collection, integration, and analysis of water quality data.

To be considered an appropriate water quality analysis tool, RF3 should provide the following functions:

- Present a working environment that allows users to explore geographic relationships between surface water features, landmark features, and data coverages.
- Allow users to select specific stream segments, including all points upstream and downstream of a given point.
- Provide tools to assist users in partitioning water quality databases into hydrologically meaningful subsets.

Reach File 3

RF3 is a hydrographic database of the surface waters of the United States. The database contains 3 million river reaches mapped at 1:100,000 scale. The source for RF3 arcs were digital line graphs (DLGs).

Attribute data for RF3 arcs include the major-minor DLG pairs, stream name, water-body type, stream order, and a unique identifying reach number. The unique reach numbers are structured in such a way as to provide a logical hydrologic framework. Reach numbers can be used to sort the database for all reaches in any specified watershed or locate all upstream or downstream reaches.

The U.S. Environmental Protection Agency (EPA) originally designed RF3 as a tabular data set. It evolved into a GIS data coverage, and EPA and the U.S. Geological Survey (USGS) will likely maintain it as a surface water mapping standard. At present, RF3 as a GIS data layer is not widely used for water quality applications.

RF3 Pilot Study: Upper Yadkin River Basin

The Upper Yadkin River basin (USGS h03040101) was selected to test RF3 water quality applications (see Figures 1 and 2). The Upper Yadkin was chosen because of the availability of water quality and stream flow data layers in that area. Also, the Upper Yadkin RF3 file contained arcs depicting lakes and double-line rivers as well as simple stream networks. These two-dimensional water features present interesting complications to network routing and path-finding.

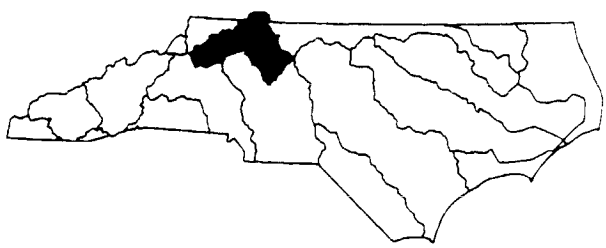

Figure 1. The Upper Yadkin River watershed, North Carolina and Virginia.

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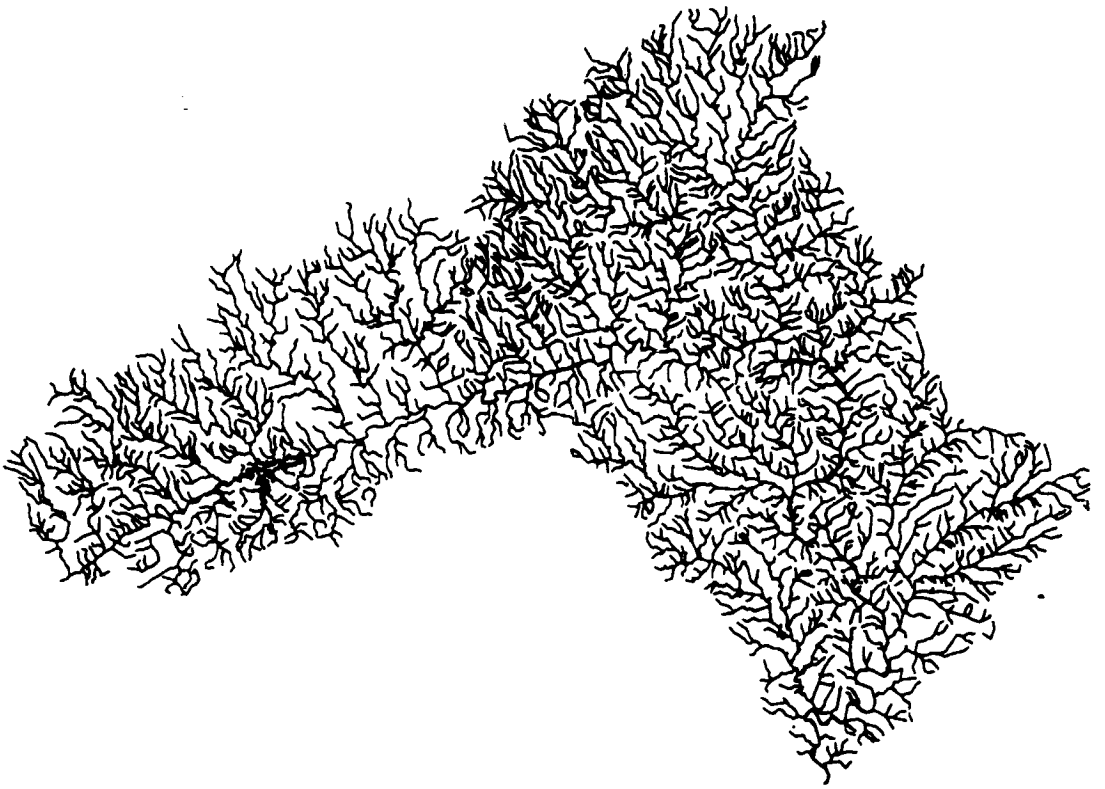

Figure 2. RF3 hydrography for the Upper Yadkin River basin.

Two forms of point source data were used in the study: National Pollutant Discharge Elimination System (NPDES) wastewater discharge points and USGS gages. The NPDES coverage includes data on the permit limits such as daily flow, dissolved oxygen, biochemical oxygen demand (BOD), and ammonia. The USGS gage coverage includes data on several flow statistics for each USGS gage in the basin. Both data layers contain information about the location of the site and stream with which it is associated.

Coverages of counties and cities were also made available for geographic orientation.

Preparing the Network

The original RF3 file received from the USGS had several topological issues that needed to be addressed before RF3 could function as a stream network. First, not all arcs were connected to each other (see Figure 3). The ARC/INFO command TRACE was used to select all connected arcs. This revealed three major blocks of connected arcs and many isolated arcs. The three major blocks were easily connected in ARCEDIT by extending the main tributary links between the blocks. Processing of the isolated arcs was not pursued for this study. Complete processing of arcs for this RF3 basin would not be difficult or time consuming, with the possible exception of the many arcs surrounding the lake. A functional network encompassing a high percentage of the arcs was not difficult to achieve, however.

The second network issue concerned the direction of the arcs. RF3 has all arcs oriented toward the top of the watershed, with the exception of one side of double-line streams. Arcs that make up double-line streams are oriented up one side of the double-line section and down the other (see Figure 3). Clearly, this complicates routing. To allow for accurate downstream routing, arcs on the downward-facing side of the stream were flipped using ARCEDIT. With all arcs in the network facing upstream, most hydrologic routes can be traced. Given the network system alone, upstream routing from double-line streams does not function properly, ignoring all tributaries on one side of the double-line stream.

Double-Line Stream Routing

Many possible solutions exist for the problems caused by double-line streams. Some involve improving the network (e.g., by adding center-line arcs down the middle of double-line streams). This would involve not only adding arcs but establishing conductivity with all tributaries. This option will involve significant topological changes to RF3. To maintain compatibility with other

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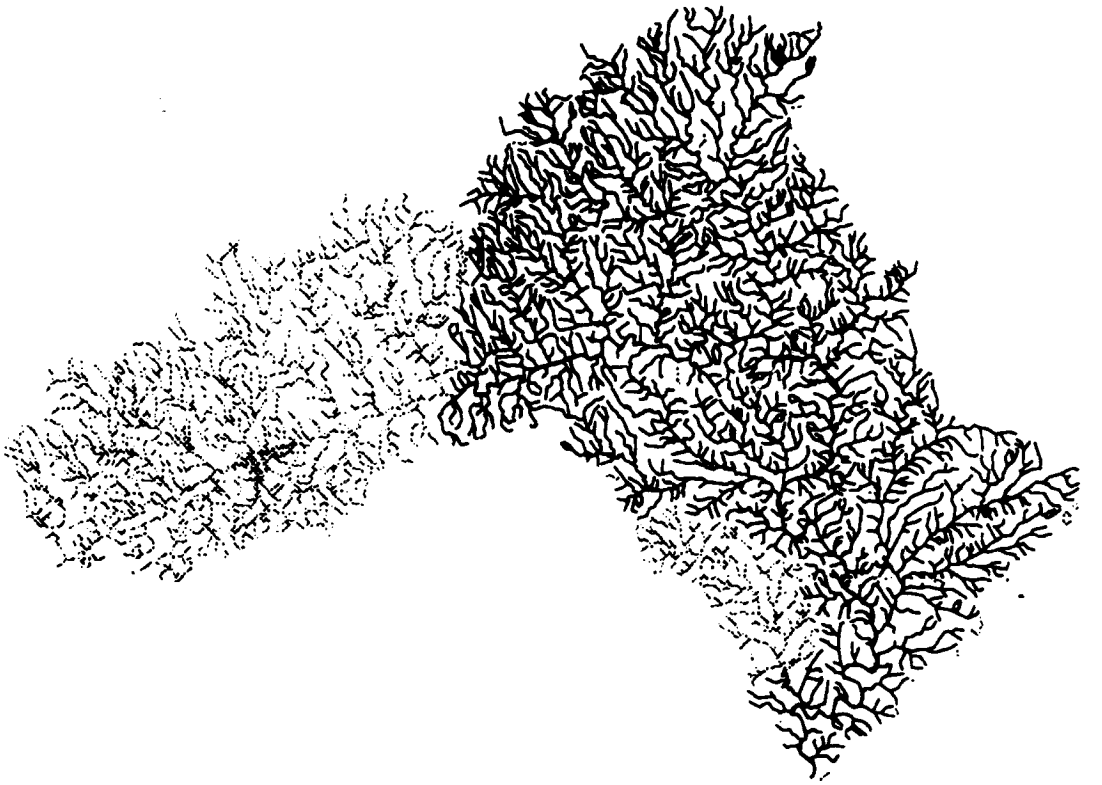

Figure 3. Original conductivity of RF3 hydrography.

RF3 and DLG sources, this option should be considered only as part of a major RF3 upgrade.

At the other end of the technological spectrum, one could simply instruct users to watch for double-line streams and select arcs from both sides of the river. Users may have trouble with this option, however, if they are not working at an appropriate scale to easily differentiate between double- and single-line streams.

A third option is to program an arc macro language (AML) to check for double-line streams and run upstream traces from both sides of the stream. The difficulty in this method is to find the appropriate starting place on both banks. The algorithm developed to do this goes as follows:

- Select stream segment and trace upstream. (Results in incomplete trace.)
- Find the minimum segment and mile of selected double-line streams.
- Unselect all double-line streams below minimum segment and mile.
- Add to selection all non-double-line streams.
- Trace from original point both upstream and downstream. (Results in completed upstream trace.)

Results and Conclusions

AMLs and menus were written that can perform upstream and downstream traces on the RF3 stream network and select data points within 500 feet of the stream. Lists of attributes can be returned to the screen. This system is easy to use and can be used to quickly identify general watersheds and water quality data points. An AML can be used to trace upstream from a double-line stream given only one point on the stream (see Figure 4). The success of these methods suggests that two-dimensional surface water features can be successfully integrated into RF3 water quality analyses.

This system could be further developed to support polygon analysis using the ARC command BUFFER. Other developments could include the procedures to write selected attributes to files and increased flexibility for the screen environment and outputs.

This pilot project demonstrates only a few of the potential applications of RF3 to water quality. Success in this pilot project suggests that RF3 is a potentially valuable water quality analysis tool. It may also be a valuable tool for demonstrating the results of water quality analyses to managers or the public.

Because RF3 will require some processing before network algorithms can be run, it is important to plan for the integration of RF3 into other GIS tools and data coverages.

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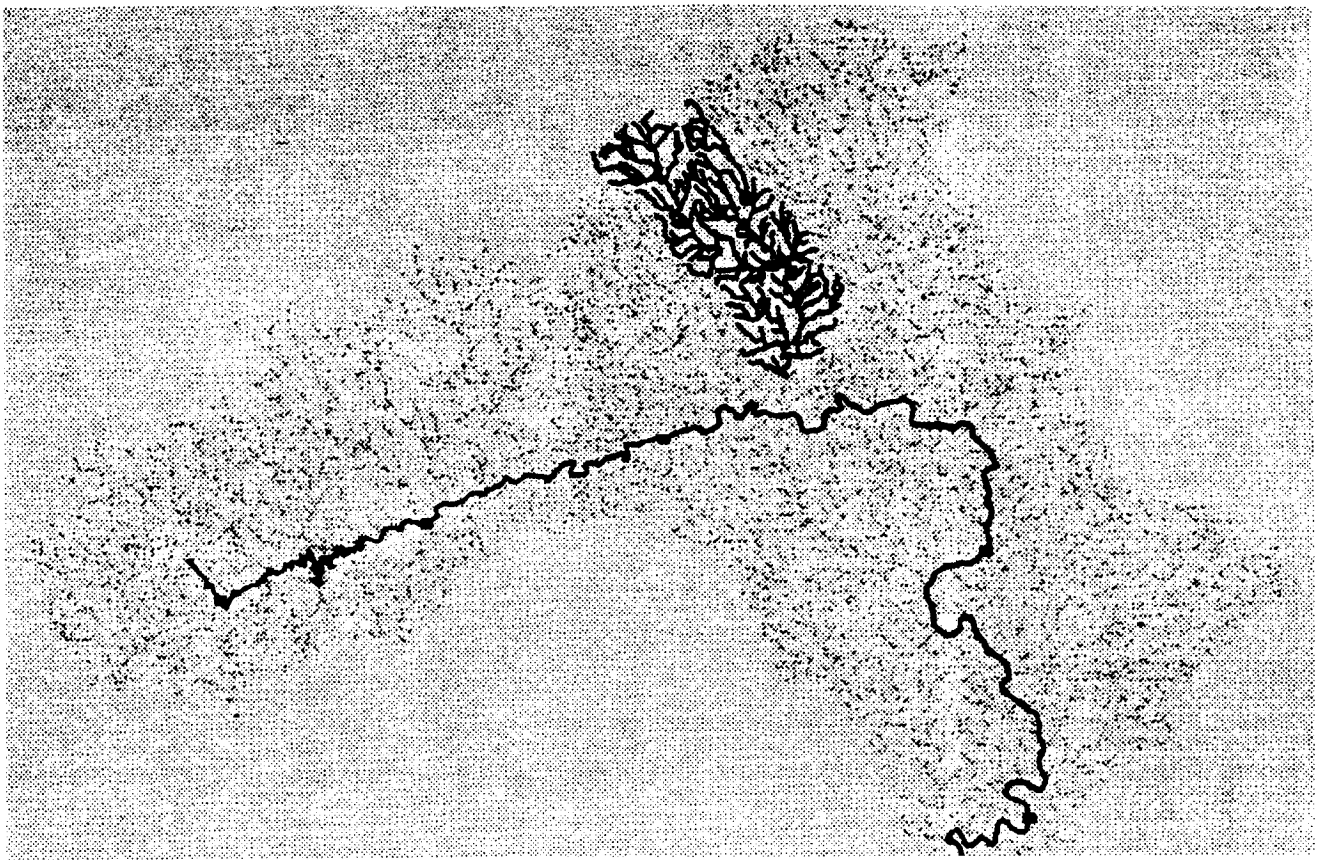

Figure 4. Upstream and downstream traces of RF3 hydrography.

Ongoing efforts to update RF3 may address some of these problems. If RF3 is to be developed into a productive water quality management tool, it is important to proceed in a way that is compatible with ongoing efforts to update RF3 and the development of new data sources.

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EPA's Reach Indexing Project—Using GIS To Improve Water Quality Assessment

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Abstract

The Waterbody System (WBS), which the U.S. Environmental Protection Agency (EPA) originally developed to support preparation of the report to Congress that Section 305(b) of the Clean Water Act requires, is a potentially significant source of information on the use support status and the causes and sources of impairment of U.S. waters. Demand is growing for geographically referenced water quality assessment data for use in interagency data integration, joint analysis of environmental problems, establishing program priorities, and planning and management of water quality on an ecosystem or watershed basis.

Because location of the waterbody assessment units is key to analyzing their spatial relationships, EPA has particularly emphasized anchoring water bodies to the River Reach File (RF3). The reach file provides a nationwide database of hydrologically linked stream reaches and unique reach identifiers, based on the 1:100,000 U.S. Geological Survey (USGS) hydrography layer.

EPA began the reach indexing project to give states an incentive to link their water bodies to RF3 and to ensure increased consistency in the approaches to reach indexing. After a successful 1992 pilot effort in South Carolina, an expanded program began this year. Working with Virginia, a route system data model was developed and proved successful in conjunction with state use of PC Reach File (PCRF), a PC program that relates water bodies to the reach file. ARC/INFO provides an extensive set of commands and tools for developing and analyzing route systems and for using dynamic segmentation.

One important advantage of the route system is that it avoids the necessity of breaking arcs; this is an important consideration in using RF3 as the base coverage in a geographic information system (GIS). Using dynamic segmentation to organize, display, and analyze water quality assessment information also simplifies use of the existing waterbody system data. Because of the variability in delineation of water bodies, however, other states used a number of different approaches. Working with these states has defined a range of issues that must be addressed in developing a consistent set of locational features for geospatial analysis.

Wider use of these data also depends upon increased consistency in waterbody assessments within and between states. Several factors complicate the goal of attaining this consistency in assessment data:

- The choice of beneficial use as the base for assessment of water quality condition.
- The historical emphasis on providing flexible tools to states.
- The lack of robust standards for assessment of water quality condition.

This paper explores possible resolutions to the problem of building a national database from data collected by independent entities.

Section 305(b) of the Clean Water Act and the Waterbody System

Background of Section 305(b)

Since 1975, Section 305(b) of the Federal Water Pollution Act, commonly known as the Clean Water Act (CWA), has required states to submit a report on the quality of their waters to the U.S. Environmental Protection Agency (EPA) administrator every 2 years. The administrator must transmit these reports, along with an analysis of them, to Congress.

State assessments are based on the extent to which the waters meet state water quality standards as measured

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against the state's designated beneficial uses. For each use, the state establishes a set of water quality criteria or requirements that must be met if the use is to be realized. The CWA provides the primary authority to states to set their own standards but requires that all state beneficial uses and their criteria comply with the 'fishable and swimmable' goals of the CWA.

Assessments and the Role of Guidelines

EPA issues guidelines to coordinate state assessments, standardize assessment methods and terminology, and encourage states to assess support of specific beneficial uses (e.g., aquatic life support, drinking water supply, primary contact recreation, fish consumption). For each use, EPA asks that the state categorize its assessment of use support into five classes:

- *Fully supporting:* meets designated use criteria.
- *Threatened:* may not support uses in the future unless action is taken.
- *Partially supporting:* fails to meet designated use criteria at times.
- *Not supporting:* frequently fails to meet designated use criteria.
- *Not attainable:* use support not achievable.

In the preferred assessment method, the state compares monitoring data with numeric criteria for each designated use. If monitoring data are not available, however, the state may use qualitative information to determine use support levels.

In cases of impaired use support (partially or not supporting), the state lists the sources (e.g., municipal point source, agriculture, combined sewer overflows) and causes (e.g., nutrients, pesticides, metals) of the use support problems. Not all impaired waters are characterized. Determining specific sources and causes requires data that frequently are not available.

States generally do not assess all of their waters each biennium. Most states assess a subset of their total waters every 2 years. A state's perception of its greatest water quality problems frequently determines this subset. To this extent, assessments are skewed toward waters with the most pollution and may, if viewed as representative of overall water quality, overstate pollution problems.

Assessment Data Characteristics

Each state determines use support for its own set of beneficial uses. Despite EPA's encouragement to use standardized use categories, the wide variation in state-designated beneficial uses makes comparing state uses an inherent problem. This affects the validity of aggregation and use of data across state boundaries. Comparably categorizing waters into use support categories also poses a problem; different states apply the qualitative criteria for use support levels in very different ways. Further limiting the utility of Section 305(b) data is that data are aggregated at the state level and questions about the use support status of individual streams cannot be resolved without additional information. While some states report on individual waters in their Section 305(b) reports, EPA's Waterbody System (WBS) is the primary database for assessment information on specific waters.

State monitoring and assessment activities are also highly variable. States base assessments on monitoring data or more subjective evaluation. The evaluation category particularly differs among states.

Waterbody System

The WBS is a database and a set of analytical tools for collecting, querying, and reporting on state 305(b) information. It includes information on use support and the causes and sources of impairment for water bodies, identification and locational information, and a variety of other program status information.

As pointed out earlier, although some states discuss the status of specific waters in their 305(b) reports, many do not. The WBS is generally much more specific than the 305(b) reports. It provides the basic assessment information to track the status of individual waters in time and, if georeferenced, to locate assessment information in space. By allowing the integration of water quality data with other related data, the WBS provides a framework for improving assessments.

WBS has significant potential for management planning and priority setting and can serve as the foundation for watershed- and ecosystem-based analysis, planning, and management. In this respect, it can play a vital role in setting up watershed-based permitting of point sources. The primary function of WBS is to define where our water quality problems do and do not exist. WBS is increasingly used to meet the identification requirement for waters requiring a total maximum daily load (TMDL) allocation. It can serve as the initial step in the detailed allocation analysis included in the TMDL process. In addition, WBS is an important component of EPA participation in joint studies and analyses. For instance, EPA is currently participating with the Soil Conservation Service (SCS) in a joint project to identify waters that are impaired due to agricultural nonpoint source (NPS) pollution. WBS can also anchor efforts to provide improved public access at the state and national levels to information on the status of their waters.

It is important to recognize that use of WBS is voluntary. Of the 54 states, territories, river basin commissions, and Indian tribes that submitted 305(b) reports, approximately

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30 used the WBS in the 1992 cycle. While submissions for the 1994 cycle are not complete, we anticipate about the same level of participation. This represents about a 60-percent rate of participation in WBS, which may be the limit for a voluntary system. This severely limits use of WBS assessment data for regional and national level analysis. If data at the national level are required, mandatory data elements, formats, and standards may be necessary.

EPA is currently attempting to achieve consistency through agreement with other state and federal agencies. The recent work of the Interagency Task Force on Monitoring offers hope for eventual consensus on the need for nationally consistent assessment data and mutually agreed upon standards for collection, storage, and transfer. The Spatial Data Transfer Standards already govern spatial data, allowing movement of data between dissimilar platforms. The Federal Geographic Data Committee provides leadership in coalescing data integration at the federal level; it provides a model for government and private sector efforts. This level of cooperation, however, has not always been present in water assessment data management. Assuming that national and regional assessment data are needed, if consistency is elusive through cooperative efforts, regulations may be necessary. Developing a national database may not be feasible without a mutual commitment by EPA and the states to using common assessment standards.

WBS was originally developed as a dBASE program in 1987. It has undergone several revisions since then, and the current Version 3.1 is written in Foxpro 2.0. The WBS software provides standard data entry, edit, query, and report generation functions. WBS has grown substantially in the years since its inception, primarily in response to the expressed needs of WBS users and EPA program offices. The program's memory requirements and the size of the program and data files, however, are of growing concern to state WBS users and the WBS program manager. Because of the wide range of WBS user capabilities and equipment, users must be equipped to support an array of hardware from high capacity Pentium computers to rudimentary 286 machines with 640 Kb of memory and small hard disks. This range makes memory problems inevitable for some users.

While WBS contains over 208 fields, exclusive of those in lookup tables, approximately 30 fields in four files provide the core data needed to comply with 305(b) requirements. These fields contain:

- Identification information for the water body.
- The date the assessment was completed.
- The status of use support for beneficial uses.
- The causes and sources of any use impairment in the water body.

The uses WBS considers are both state-designated uses and a set of nationally consistent uses (e.g., overall use, aquatic life support, recreation) specified in the 305(b) guidelines. The other essential piece of information is the geographic location of the water body, which the remainder of this paper discusses in detail.

Significant differences exist in the analytical base as well as in assessments. EPA provided little initial guidance on defining water bodies; therefore, states vary widely in their configurations of water bodies. Water bodies are supposed to represent waters of relatively homogeneous water quality conditions, but state interpretation of this guidance has resulted in major differences in water-body definition.

Initially, many states developed linear water bodies, and these were often very small. The large number of water bodies delineated, however, created significant difficulties in managing the assessment workload and were not ideal in the context of the growing need for watershed information. Some states, such as Ohio, developed their own river mile systems.

As discussed below, some states indexed their water bodies to earlier versions of the reach file, and therefore, the density of the streams these water bodies include is fairly sparse. Recently, many states have redefined their water bodies on the basis of small watersheds (SCS basins, either 11-digit or 14-digit hydrologic unit codes [HUCs]).

Locating water bodies geographically is a necessary prerequisite to assessing water quality on a watershed or ecosystem basis. The WBS has always included several locational fields, including county name and FIPS, river basin, and ecoregion. These fields have not been uniformly populated, however. One of the WBS files includes fields for the River Reach File (RF3) reaches included in the water body. While a few states had indexed their water bodies to older versions of the reach file (RF1 and RF2), however, no state had indexed to RF3 until 1992.

In 1992, EPA initiated a demonstration of geographic information system (GIS) technology in conjunction with the South Carolina Department of Health and Environmental Control. This project involved:

- Indexing South Carolina's water bodies to RF3.
- Developing a set of arc macro languages (AMLs) for query and analysis.
- Producing coverages of water quality monitoring stations and discharge points.
- Using GIS tools in exploring ways to improve water quality assessments.

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South Carolina has defined its water bodies as SCS basins.

The results have been very encouraging. First, South Carolina took the initial coverages and decided they needed much more specificity in their use support determinations and their mapping of the causes and sources of impairment. As a result, they mapped these features down to the reach level. Next, they decided that they needed better locational information, so they used global positioning satellite receivers to identify accurate locations for discharges and monitoring stations. They then used GIS query and analysis techniques to relate their monitoring and discharge data to their water quality criteria. South Carolina is using GIS to actively identify water quality problems and improve their assessments.

In 1993, EPA worked cooperatively with several states to index their water bodies to the reach file. Virginia, the next state to be indexed, demonstrated the successful use of PC Reach File (PCRF) software (described later in this paper) for indexing water bodies to the reach file. Ohio and Kansas also are essentially complete. Each of these states required a somewhat different approach than Virginia. The need for flexibility in dealing with states on reach indexing issues is essential. Existing waterbody delineations often represent considerable investment; therefore, EPA must provide the capability to link the state's existing assessment data to the reach file in order to encourage state buy-in.

Figure 1 shows the results of Ohio's indexing of a typical cataloging unit (CU). Figure 2 reflects part of the output of the Kansas work. We can link use support, cause, and source data to each of these water bodies now. In the future, we hope to map these attributes at a higher level of resolution, down to the reach segment level. GIS has proven to be a useful assessment tool. With higher resolution, it should prove to be even more helpful in identifying water quality problems, picking up data anomalies, and assessing management actions, strategies, and policies. This entire process has taught us much and has strengthened enthusiasm for place-based management.

The Reach Indexing Project—Georeferencing the Waterbody System

Purpose and Overview

The reach indexing project is designed to locate water bodies using RF3 as an electronic base map of hydrography and to code RF3 reaches with the specific waterbody identifier (WBID). After linking water bodies to their spatial representation, they can be queried and displayed with assessment data located in WBS files.

Reach indexing includes several steps. First, the state must supply waterbody locations and WBIDs. The next step entails developing a set of procedures for indexing. Finally, the coded RF3 data must be produced.

Input data to the indexing process includes:

- A list of valid WBIDs. In most cases, the state has already input these identification numbers to the WBS.
- Information about the location of each water body. Locational information may be found in marked-up paper maps showing waterbody locations or electronic files from WBS containing waterbody indexing

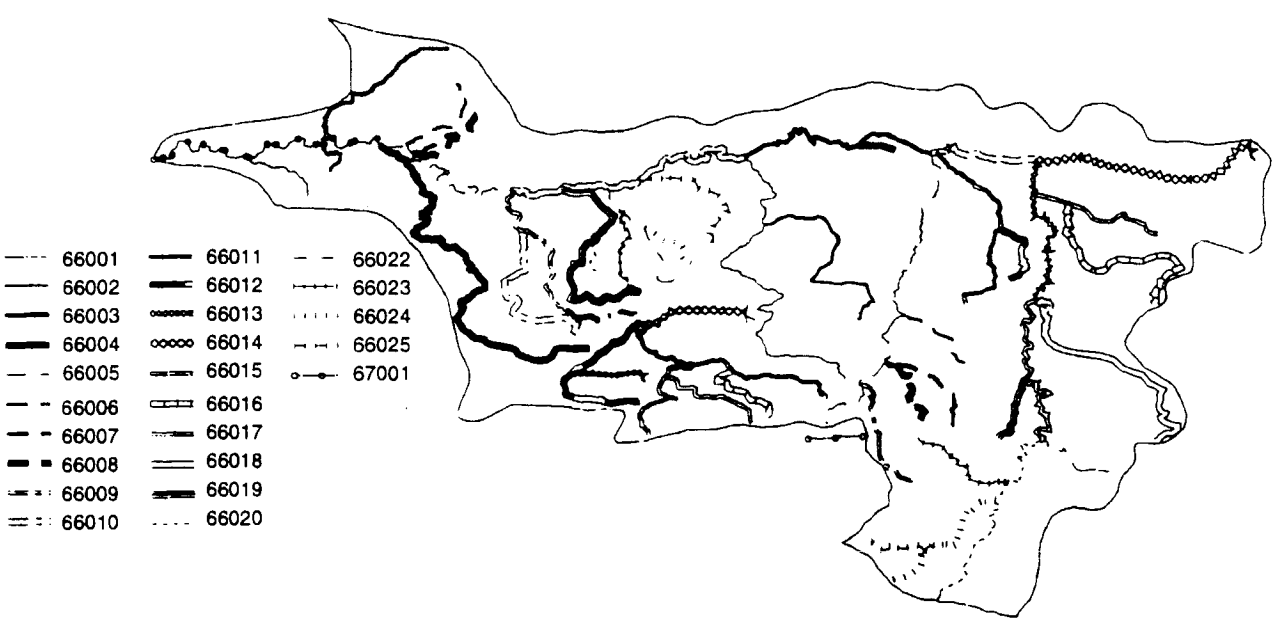

Figure 1. State of Ohio water bodies in CU 04100008.

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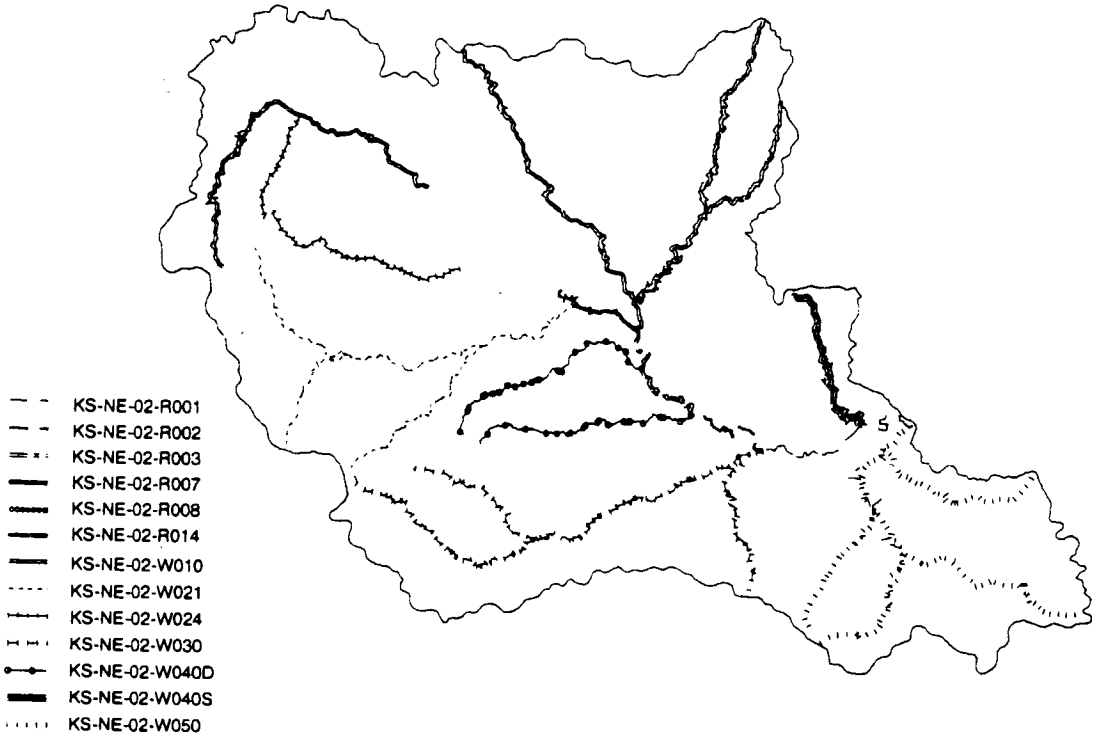

Figure 2. State of Kansas water bodies in CU 11070202.

expressions (discussed later), or it may be embedded in the WBID itself.

- A complete set of RF3 data for the state being indexed.

Depending on the type of information the state supplies, procedures used to index water bodies can be almost fully automated, semiautomated, or completely manual.

The final result of the indexing processes is a set of RF3 coverages that contain a WBID attribute. This product allows querying and displaying of assessment data, which is collected and stored by water body, in a GIS environment.

The Reach File Database

The reach file is a hydrographic database of the surface waters of the continental United States. Elements within the database represent stream segments. The elements were created for several purposes:

- To perform hydrologic routing for modeling programs.
- To identify upstream and downstream connectivity.
- To provide a method to uniquely identify any particular point associated with surface waters.

The unique reach identifier has succeeded in associating other EPA national databases, such as STORET, to surface waters. Any point within these databases can be associated with and identified by a specific location on any surface water element, such as a reservoir, lake, stream, wide river, or coastline.

There are three versions of the reach file. The first was created in 1982 and contained 68,000 reaches. The second version, released in 1988, doubled the size of Version 1. The third version (RF3) includes over 3,000,000 individual reach components.

The base geography of RF3 is derived from U.S. Geological Survey (USGS) hydrographic data (1:100,000 scale) stored in digital line graph (DLG) format. Unlike DLG data, which are partitioned by quad sheet boundaries, RF3 data are partitioned by CU. A CU is a geographic area that represents part or all of a surface drainage basin, a combination of drainage basins, or a distinct hydrologic feature. The USGS uses CUs for cataloging and indexing water-data acquisition activities.

The continental United States comprises over 2,100 CUs. CUs are fairly small; for example, 45 units fall partially or completely within the state of Virginia (see Figure 3).

RF3 is a powerful data source used in hydrologic applications for many reasons, including the following:

- RF3 has spatial network connectivity that topological upstream/downstream modeling tools use.

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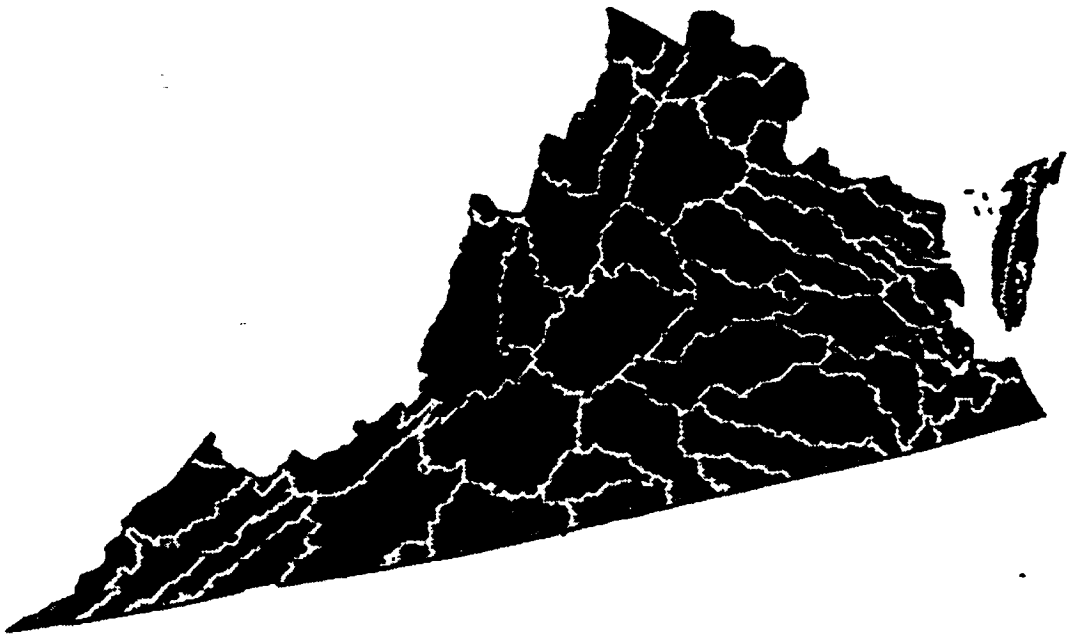

Figure 3. Cataloging units in Virginia.

- RF3 has attributes that describe connectivity, which offers the ability to accomplish upstream/downstream navigation analytically (without topological networking).
- RF3 has a simple and consistent unique numbering system for every stream reach in the United States.
- RF3 has built-in river mileage attributes that describe upstream/downstream distances along river reaches.

Use of RF3 in the Indexing Process

When importing Reach File data from EPA's mainframe computer, an arc attribute table (AAT) is automatically built for each RF3 coverage. The AAT contains the standard AAT fields, plus the items found in Table 1.

The CU item stores the USGS CU number of this piece of RF3. Every arc in the coverage has the same value for CU.

The SEG item stores the number of the stream segment to which the particular arc is assigned. SEG numbers start at 1 and increase incrementally by 1 to 'N' for each CU. A SEG could represent all the arcs of a mainstream, the arcs of a tributary, or piece of a mainstream or tributary. SEG numbers were defined in the production of RF3.

MI stores the marker index for each particular arc. The MI resembles a mile posting along a stream. In reality, the MI field does not truly measure mileage along the RF3 stream network. It does, however, represent a method of producing a unique identifier (in combination with the CU number and the SEG number) for every reach in the United States (see Figure 4).

Table 1. Fields Found in Arc Attribute Table

12070104-ID	CU	SEG	MI	UP	DOWN
1	12070104	1	0.00	-1	0
2	12070104	1	1.30	-1	0
3	12070104	1	2.10	-1	0
4	12070104	2	0.00	-1	0
5	12070104	3	0.00	-1	0
6	12070104	3	1.15	-1	0
7	12070104	4	0.00	-1	0

Together CU, SEG, and MI uniquely identify every arc in RF3 nationwide. These three items are combined in the redefined item called RF3RCHID. This provides a powerful scheme for consistently identifying locations along streams everywhere in the country.

Along with the AAT file, a second attribute file is automatically created for RF3 coverages. This file is always named COVER.DS3. The DS3 file stores a wealth of information about arcs in the coverage. Some of the important fields in the DS3 file contain:

- Upstream and downstream connectivity for navigating along reaches.
- Codes to describe the type of reach (e.g., stream, lake boundary, wide river).
- DLG major and minor attributes.

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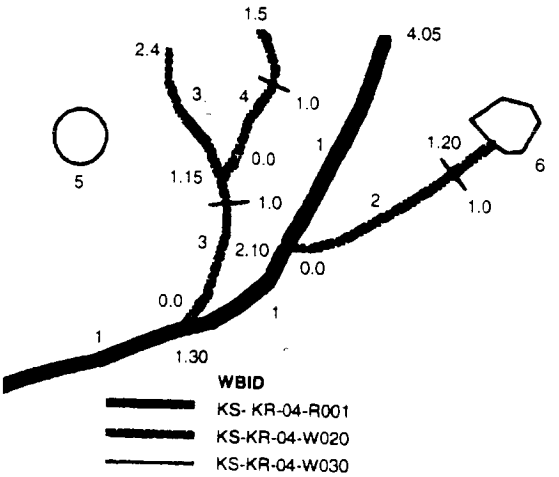

Figure 4. RF3, SEG, and MI data elements.

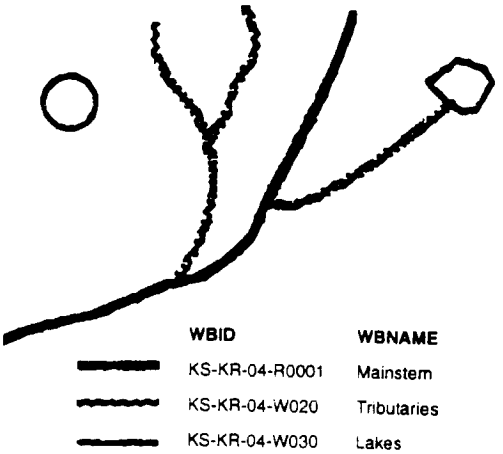

Figure 5. Potential definitions of water bodies.

Waterbody Locations

Because states define water bodies, they provide the only information on waterbody location. South Carolina was indexed to RF3 in 1992, followed by Virginia. Virginia indexed its water bodies using the PCRF program instead of indexing in a GIS environment with ARC/INFO.

PCRF is a PC-based system that indexes water bodies and locates other assessment data from WBS. PCRF stores the definitions of water bodies (including their location) in a file that is linked to other WBS database files that contain information about the assessment status and quality of the waters.

A water body is a set of one or more hydrologic features, such as streams, lakes, or shorelines, that have similar hydrologic characteristics. Water bodies are the basic units that states use to report water quality for CWA 305(b) requirements. Depending on the state's assessment goals and resources, water bodies can be defined in several ways, including (see Figure 5):

- All streams within a watershed
- All lakes and ponds within a watershed
- Sets of streams with similar water quality conditions

PCRF stores locational data for a water body with a unique WBID. WBS uses this WBID as a common field to relate the water body's definition and location to descriptive data about the water body's assessment status and quality. The two most important files used in PCRF are the SCRF1 and SCRF2 files.

The SCRF1 file simply lists the unique water bodies by state. Table 2 offers an example. The most relevant data for reach indexing in this file are the WBID, WBNAME, and WBTYPE, as defined by the state. The WBID, as stated, is a unique identifier for each water body the state has defined. The WBNAME stores a verbal description of the water body. Finally, the WBTYPE defines the type of water body; for example, R is for river, L is for lake.

The SCRF2 file contains an explicit definition of each water body. Because of the complexity involved in defining water bodies, this file may include more than one record for each water body. The SCRF2 file can be considered a waterbody definition *language* because it contains specific codes, attributes, and keys that can be converted into specific reaches on the RF3 data, if read properly (see Table 3). The WBBEGIN and WBEND fields contain explicit CU, SEG, and MI attributes to define the location of the starting point and ending point for the water body. The WBDIR field contains an attribute that describes whether to go upstream or downstream from the WBBEGIN to the WBEND. In addition, a blank WBEND field denotes that the water body should include all upstream or downstream reaches (depending on the WBDIR) of the WBBEGIN reach.

Virginia used PCRF to create an SCRF2 file that contains reach indexing expressions for all of their defined water bodies. ARC/INFO macros were then written to process this file and expand the expressions into the set of specific arcs that compose each water body. The macros will be described in more detail later.

Table 2. Example of SCRF1 File Data

WBID	WBNAME	WBTYPE	WBSIZE
KS-KR-04-R001	KANSAS RIVER	R	15.20
KS-KR-04-W020	LOWER WAKARUSA RIVER	R	61.60
KS-KR-04-W030	MUD CR	R	39.43
KS-KR-04-W040	CAPTAIL CR	R	15.63

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Table 3. Example of SCRF2 File Data

WBID	WBDIR	WBBEGIN	WBEND	RFORGFLAG
KS-KR-04-R001	U	10270104001 0.00	1027010400115.20	2
KS-KR-0R-W020	D	10270104005 10.80	10270104005 0.00	2
KS-KR-04-W030	U	10270104059 12.05	10270104007 8.10	3
KS-KR-04-W040	U	10270104038 0.00		3

States that have not already generated indexing expressions in PCRF must provide locations in some other way. The most basic method is for the state to supply a set of 1:100,000 USGS quad sheets that they have marked up with locations of each water body. The maps can be used in conjunction with a digitizer to manually select the appropriate RF3 reaches and code them with the WBID.

The state of Ohio created a GIS database of its river reaches several years ago. The GIS coverage is representational in nature. The stream reaches are 'stick-figures' only. Generally, they fall along the paths of the actual streams, but they are schematic in nature and do not show the true shape of streams. The GIS layer, however, contains the attributes of Ohio's stream reach numbering system, which is used to identify water bodies as well. Ohio's river reach coverage contains information on the locations of water bodies and is being manually conflated to transfer the WBIDs to RF3. The conflation process will be covered later in this paper.

The state of Kansas had previously defined its water bodies on RF2, the precursor to RF3. Kansas defined some indexes by a set of RF3 SEG numbers in a CU and some by the RF3 reaches in a small watershed polygon within a CU. The locations were, in effect, defined within the WBID itself.

Indexing Procedures

Procedures developed for performing waterbody indexing include automated, semiautomated, and manual systems.

Automated Indexing Procedures

As stated, Virginia used PCRF to perform the indexing operation. The state delivered an SCRF2 file containing indexing expressions for all of its water bodies. AML programs were created to read the SCRF2 file and select the reaches specified by each indexing expression. The selected sets of reaches were then coded with the appropriate WBID. The macros were designed to run on one RF3 CU at a time, so the operator specified runs of up to 10 CUs at a time. The macros had to accommodate indexing expressions that included:

- Select reaches upstream of a specified location.
- Select reaches on a reach-by-reach basis.
- Select reaches within a given polygon area.
- Select shorelines of lakes or ponds given latitude and longitude coordinates.
- Select reach downstream from a specified location.

Kansas water bodies were also indexed through an automated process. Kansas supplied an ARC/INFO coverage of small watershed polygons (sub-CU polygons) containing a watershed identifier. The state's WBID contained all other information necessary to determine the RF3 CU and the set of reaches making up each water body. An example of a Kansas WBID is KS-KR-02-W030. This is explained by the following:

- KS refers to the state. All WBIDs in Kansas begin with KS.
- The second component (in this case KR) is an abbreviation of the basin in which the water body falls. KR indicates that this water body is in the Kansas-Lower Republican River basin.
- The third component contains the last two digits of the eight-digit CU number. Although basins comprise several CUs, the last two digits of each CU in a basin are unique; therefore, between the basin (e.g., KR) designation and the last two digits of the CU (e.g., 02), the complete eight-digit CU number in which the water body falls is defined.
- The next letter (in this case W) denotes whether the water body is defined by a watershed polygon (W), an RF2 SEG (R), or a lake or pond shoreline (L).
- Finally, the WBID ends with the number of the polygon (in this case 030) that contains the reaches for the water body in the watershed coverage.

The completed macros could index the entire state in a single run provided that all the WBIDs were contained in single file.

In all cases, Kansas has indexed to RF2 reaches. Only RF3 reaches originally created in RF2 production, therefore, are coded with a WBID.

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Manual Indexing Procedures

Because Ohio already has a coverage of river reach codes, WBIDs from this coverage had to be transferred to the RF3 reaches they represent. This entailed using a manual conflation process. The operator displayed a CU of RF3 along with the Ohio river reach system for the same area. In a simple process of 'pointing and clicking,' the operator first selected an Ohio river reach arc, then the RF3 arcs that seemed to coincide. As each RF3 arc was selected, it was coded with the WBID of the previously selected Ohio river reach arc.

Other states that have no means of describing water bodies in electronic files may have to mark up paper maps to show waterbody locations. These maps can then be used in a manual process of selecting RF3 reach and coding them with WBIDs either in ARC/INFO or in PCRF.

Using the Route System Data Model To Store Water Bodies

Because water bodies can be defined as noncontiguous sets of arcs and portions of arcs, a robust linear database model is necessary to model these entities. ARC/INFO's route system data model seems well suited for this application. The route system data model allows one to group any set of arcs or portions of arcs into routes. Each route is managed as a feature in itself. Attributes of water bodies are stored in a route attribute table (RAT) and relate to all the arcs defined as the water body. Figure 6 helps illustrate the route system model.

Each route comprises one or more arcs or sections of arcs. ARC/INFO manages the relationship between arcs and routes in the section table (SEC). The structure of the SEC, which is an INFO table, is defined in Table 4. Table 5 reflects how the sections that make up the above routes would appear.

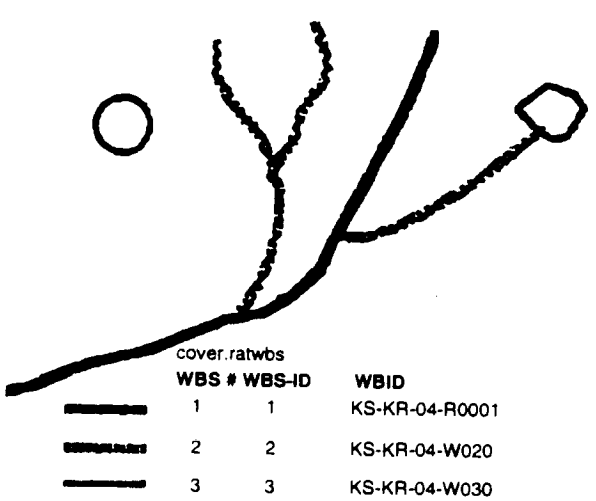

Figure 6. The route attribute table containing waterbody data.

Table 4. Definition of Structure of SEC INFO Table

ROUTELINK#	The route upon which the section falls
ARCLINK#	The arc upon which the section falls
F-MEAS	The measurement value at the beginning of the section
T-MEAS	The measurement value at the end of the section
F-POS	The percentage of the distance along the arc at which the section begins
T-POS	The percentage of the distance along the arc at which the section ends
SEC#	The internal identifier of the section
SEC-ID	The user identifier of the section

Table 5. How Sections Appear in SEC INFO Table

ROUTE-LINK#	ARCLINK#	F-MEAS	T-MEAS	F-POS	T-POS	SEC#	SEC-ID
1	1	0	1.30	0	100	1	1
1	2	1.30	2.10	0	100	2	2
1	3	2.10	4.05	0	100	3	3
2	4	0	1.20	0	100	4	4
3	5	0	1.15	0	100	5	5
3	6	1.15	2.4	0	100	6	6
3	7	0	2.5	0	100	7	7

Representing Water Bodies as Routes

ARC/INFO offers several ways of grouping sets of arcs into discrete routes. One can use ARCEDIT to select a set of arcs to group them into a route, or ARCSECTION or MEASUREROUTE in ARC to group arcs into routes. The method described here uses the MEASUREROUTE command. This method requires that the AAT or a related table has an attribute containing the identifier of the route to which an arc should be assigned. In the application the authors employed, they converted the SCRF2 file into an INFO table containing, for each arc in the coverage, the RF3RCHID of the arc and the WBID to which the arc should be assigned. The WBID item is used to group arcs into routes. One route exists for each unique WBID. Table 6 illustrates the table used in the MEASUREROUTE command method. This table is related to the AAT of the RF3 coverage by the RF3RCHID.

An RAT is automatically created for the coverage, which now can be related to other WBS assessment files for display and query. Figure 4 illustrates the RAT. The most important characteristic of the file is that it has only one record for each water body. This simplifies the display and query of water bodies based on water quality data.

Using EVENTS for Subwaterbody Attributes

Water bodies, as states define them, often constitute a gross aggregation of the water in an area. States often have more specific data about particular stretches of streams within a water body. A system is needed to

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Table 6. Table Used in MEASUREROUTE Command Method To Group Arcs Into Routes

$RECNO	RF3RCHID	WBID
1	10270104 1 0.00	KS-KR-04-R0001
2	10270104 1 1.30	KS-KR-04-R0001
3	10270104 1 2.10	KS-KR-04-R0001
4	10270104 2 0.00	KS-KR-04-W020
5	10270104 3 0.00	KS-KR-04-W020
6	10270104 3 1.15	KS-KR-04-W020
7	10270104 4 0.00	KS-KR-04-W020
8	10270104 5 0.00	KS-KR-04-W030
9	10270104 6 0.00	KS-KR-04-W030

query and display data at the subwaterbody level. ARC/INFO's dynamic segmentation tools and event tables are useful for this application. Once water bodies have been defined and reporting methods have been set up based on those water bodies, the task of redefining them is cumbersome.

Event tables can help to keep these waterbody definitions yet still offer the ability to store, manage, and track data at the subwatershed level. Event tables are simple INFO files that relate to route systems on coverages. This concept and data structure can work in conjunction with the predefined waterbody system. We have already seen how a route system called WBS is created in RF3 to group arcs into waterbody routes. This works quite well when displaying water bodies and querying their attributes. A route system based on the WBID cannot, however, act as an underlying base for subwaterbody events because the measures used to create the WBS route system are not unique for a particular route. For example, in the route depicted in Figure 7, three locations are defined as being on WBID KS-KR-04-W020 and having measure 1.0.

The mileage measurements along SEG, however, are always unique (see Figure 8). To use EVENTS, therefore, a second route system must be created based on the RF3 SEG attribute, which provides a unique code for each CU.

The ARCSECTION command, instead of the MEASUREROUTE command, is used to create the SEG route system. This is because the measurement items (MI on the AAT and UPMI on the DS3) already store the summed measures along particular SEGs. Table 7 lists the contents of the resulting RAT table.

Because the name of the route system is SEG, the SEG# and SEG-ID are the names of the internal and user IDs. The SEG item contains the actual SEG number in the RF3 coverage. Because the SEG numbers for each RF3 CU coverage start at 1 and increase incrementally by 1, the SEG item looks much like the SEG-ID and SEG#.

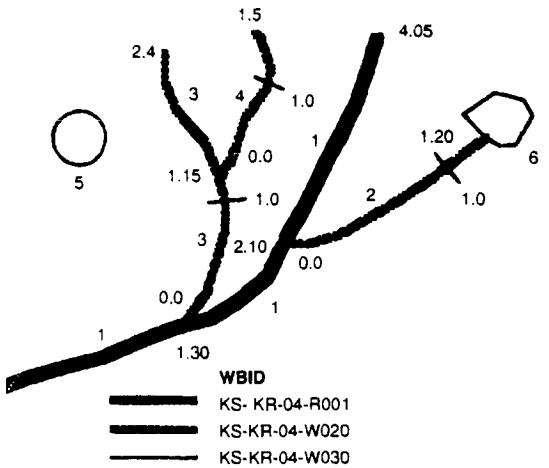

Figure 7. Measurements along SEGs.

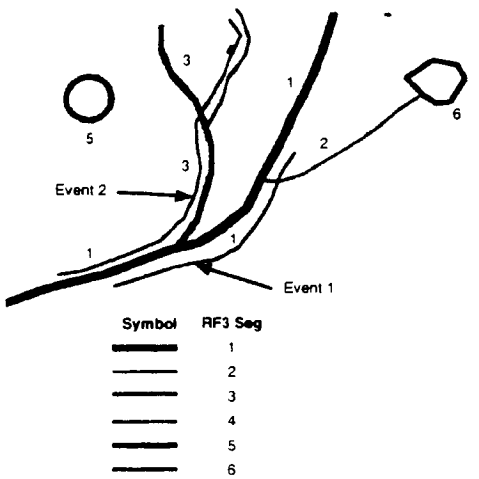

Figure 8. Events located on RF3 data.

Table 7. Route Attribute Table

SEG-ID	SEG#	SEG
1	1	1
2	2	2
3	3	3

Event tables contain a key item, the WBID or SEG, to relate them to the appropriate route system (see Figure 8). They also contain locational information on where to locate the events on the route (either WBID to indicate the water body on the WBS route or SEG to identify the route in the SEG route system). Separate event tables can then relate use support, causes, and sources as linear events. FROM and TO store the starting and

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ending measures for each event. Using event tables allows us to apply many useful cartographic effects (e.g., hatching, offsets, text, strip maps). Events can be queried both in INFO and graphically. Event data can help in producing overlays of two or more event tables.

An event table can display use support information (see Table 8). WBS users can update their event tables using RF3 maps supplied by EPA without having proficiency in ARC/INFO. ARC/VIEW2 is expected to support events and route systems. This will give users powerful tools for spatial query of assessment data. Developing event tables would also display and query data on the causes and sources of use impairment. These events can be offset and displayed to show the areas of interaction. More permanently, preparing line-on-line overlays can show intersections and unions.

An alternative approach is to use an EVENT-ID as a unique identifier for each event. The SEG field stores the number of the route (SEG) upon which the event occurs. FROM and TO store the beginning and end measures along the route upon which the event occurs. WBID contains the identifier of the water body upon which the event occurs (see Table 9). An event can occur within a single SEG, across two or more SEGs, within a single water body, or across two or more water bodies.

Additional attribute tables can be created to store descriptive attributes for each event. These tables would resemble the SCRF5 and SCRF6 files except that instead of using the WBID to relate to a water body, a field called 'EVENT-ID' would link the use, cause, and source data to a particular event (see Table 10).

Both approaches offer some advantages. In either case, they allow us to map our water quality assessment data and communicate it in a meaningful and useful way.

Table 8. Event Table That Reflects Use Support Information

SEG	FROM	TO	WBID	USE	USE SUPPORT
1	0.80	1.30	KS-KR-04-R0001	21	Fully
1	1.30	2.10	KS-KR-04-R0001	21	Partial
1	2.10	2.31	KS-KR-04-R0001	21	Not supported
1	0.50	1.30	KS-KR-04-R0001	40	Threatened
3	0.00	1.15	KS-KR-04-W030	21	Fully
4	0.00	2.5	KS-KR-04-W040	40	Not supported

Table 9. Using EVENT-ID as a Unique Event Identifier

EVENT-ID	SEG	FROM	TO	WBID
1	1	0.80	1.30	KS-KR-04-R0001
1	1	1.30	2.10	KS-KR-04-R0001
2	4	0.00	2.5	KS-KR-04-W040

Table 10. Using EVENT-ID To Link Use, Cause, and Source Data to an Event

EVENT-ID	ASCAUSE	ASSOURC
1	900	1200
1	-9	1100
1	0500	1100
2	1200	9000
2	0900	8100

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Environmental Management Applications

Ecological Land Units, GIS, and Remote Sensing: Gap Analysis in the Central Appalachians

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Abstract

The gap analysis team in West Virginia is assessing the state's natural communities as part of a nationwide, comprehensive planning effort. Underrepresented or unrepresented habitats represent gaps in the present network of conservation lands and conservation activities. After identifying these gaps, we can assess whether our current management direction will maintain natural diversity and will prevent additional species from being classified as threatened or endangered.

The relationship between vegetation and ecological variables serves as the basis for classifying ecological land units. To characterize the ecological land units, many layers of physical data can be integrated in a geographic information system (GIS). Satellite imagery and videography map existing conditions over the state. The existing vegetation is classified to reflect physiognomic and floristic elements to correlate with vertebrate and butterfly habitat requirements. This correlation of vegetation and wildlife habitat creates mappable habitat types. Analysis of these habitat types with land-ownership data indicates where the species-rich areas occur in the landscape and whether the most species-rich areas are protected.

Introduction

To respond to the urgency of habitat loss and its effect on species diversity, scientists must implement a methodology for rapid assessment and documentation of natural communities at a scale pertinent for regional management activities.

Geographic information systems (GIS) and remote sensing support the development of ecological land classifications over large regions. GIS-based mapping of ecological land classes allows users to combine and display environmental variables for spatial modeling and refinement of ecological land units (1).

The Gap Analysis Project is a comprehensive planning effort for land conservation in the United States. The objective of the Gap Analysis Project is to identify species, species-rich areas, and vegetation types underrepresented or unrepresented in existing biodiversity management areas. Unprotected communities are the gaps in the conservation strategy. The Gap Analysis Project is not merely identifying communities with the largest number of species; its ultimate goal is to identify clusters of habitats that link the greatest variety of unique species.

Local areas with considerable diversity of habitat or topography usually have richer faunas and floras (2). Nature reserves, which incorporate a variety of habitats, may be the best guarantee for long-term protection of biodiversity. By protecting species-rich regions, we can reduce the enormous financial and scientific resources needed to recover species on the brink of extinction.

The West Virginia Gap Analysis Project began in 1991. The objective is to map existing vegetation and to use that as the foundation to model potential distribution of vertebrate and butterfly species. High cost precludes intensive field inventory and monitoring of wildlife. Therefore, habitat modeling is critical to predict wildlife species composition and potential ranges over the various landscape types of West Virginia. Lastly, the vegetation map will provide a record of the existing habitat to use in monitoring changes due to human activities and natural disturbances.

Pilot Study Area

Distribution of wildlife and plant communities will be modeled for the entire state. Initially, we will focus on a smaller pilot study area. This region includes approximately 50,000 hectares in the central Appalachian Mountains and spans several physiographic provinces and vegetative communities. Generally, soils in the pilot study area are of two kinds: acidic soils that develop a

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clay horizon from extensive leaching over time and younger soils that are found on steep slopes and where environmental conditions, such as cold climate, limit soil development (3, 4).

The vegetation types include spruce-fir, oak-pine, high elevation bogs, northern hardwoods, Appalachian oak, mixed-mesophytic, open heath barrens, and grass balds. The mixed-mesophytic, the most diverse in West Virginia, lies primarily west of this area, but localized stands do occur in the lower elevations. Cover types within the Appalachian oak and mixed-mesophytic types are not discrete and will be difficult to delineate.

The pilot study area includes a variety of land uses, such as residential, commercial, industrial, mining, and agriculture. Portions of the Monongahela National Forest in this area are the Fernow Experimental Forest, and the Otter Creek and Laurel Forks Wilderness Areas.

Methods

The following discussion describes the methods formulated and data compiled for the West Virginia Gap Analysis Project.

Describe Ecological Land Units With Existing Vegetation

Davis and Dozier (1) note that a landscape can be partitioned by ecological variables, which contributes to an ecological land classification. This process is applied frequently to analysis and mapping of natural resources. Davis and Dozier classified vegetation in California based on the documented associations of vegetation with terrain variables. They based this approach on the assumptions that the arrangement of natural landscape features is spatially ordered by an ecological interdependence among terrain variables and that actual vegetation is a reliable indicator of these ecological conditions. Similar documentation exists for the distribution of vegetation types in West Virginia, and the gap analysis team is proceeding along a similar course.

West Virginia lies in two major provinces, the Eastern Broadleaf Forest and the Central Appalachian Broadleaf Forest-Coniferous Forest-Meadow Provinces (5). Within these provinces are several broad vegetation types. The gradient diagram in Figure 1 (6) illustrates the range of these types. The vertical axis represents elevation in feet. Three vegetation types emerge distinctly along the elevation gradient. The horizontal axis is not quite as explicit. This gradient spans moist, protected slopes to dry, exposed ridgetops, and the vegetation types are much less distinct. Drier oak and pine types occur almost exclusively on exposed ridgetops. The vegetation types along the horizontal axis are the mixed mesophytic forest association of the Allegheny and Cumberland Mountains and can have 20 to 25 overstory and understory species per hectare in North America (7).

The distribution of the vegetation along gradients such as elevation and soil moisture lends itself to a GIS analysis. Physical data such as elevation and soil moisture regime can be incorporated into a GIS. These layers of information can be manipulated graphically or mathematically to model the spatial distribution of vegetation types or to provide useful ancillary data for classification of satellite imagery (see Figure 2). Much of this data is digital and can be used to substantially reduce the time required to develop a database. To standardize output, members of the national gap team have specified ARC/INFO as the software to generate final products.

Classify Satellite Imagery To Create a Map of Current Distribution

Remote sensing provides an effective means to classify forests, and the Gap Analysis Project has successfully used it in the western United States (1, 8, 9).

The Gap Analysis Project is using Landsat Thematic Mapper imagery in all states to standardize the baseline information. The hypothesis is that the spectral data from the imagery is related to the distribution of the ecological land units and land use across the landscape. The data include all spectral bands, except thermal, for the entire state. The West Virginia project is using two seasons of data, spring and fall. Temporal changes, which record phenologic variation in the deciduous species, enhance classification accuracy. The spectral resolution is 30-meter pixels. This is equivalent to approximately 1/6 hectare (1/2 acre). Our final product will be a series of 1:100,000 maps. The minimum mapping unit is 40 acres.

The mountainous terrain in the Appalachian Mountains offers disadvantages and advantages for using remotely sensed data. Irregular topography can cause inconsistencies in the spectral data that diminish the classification accuracy. Similar cover types may have different spectral signatures; for instance, if one stand is in sunlight and the other is shaded. Also, phenology can vary due to microclimatic influences. Conversely, topographic features influence the distribution of vegetation types, and ancillary data, such as digital elevation models (DEMs), enhance classification results of the imagery. The West Virginia gap analysis team selected the following strategy for image classification.

1. Stratify the imagery using ecological units based on a hierarchical scheme. Bauer et al. (10) found that an initial stratification of physiographic regions was necessary to reduce the effect of broadscale environmental factors caused by changes over latitude. Therefore, stratification enhanced the

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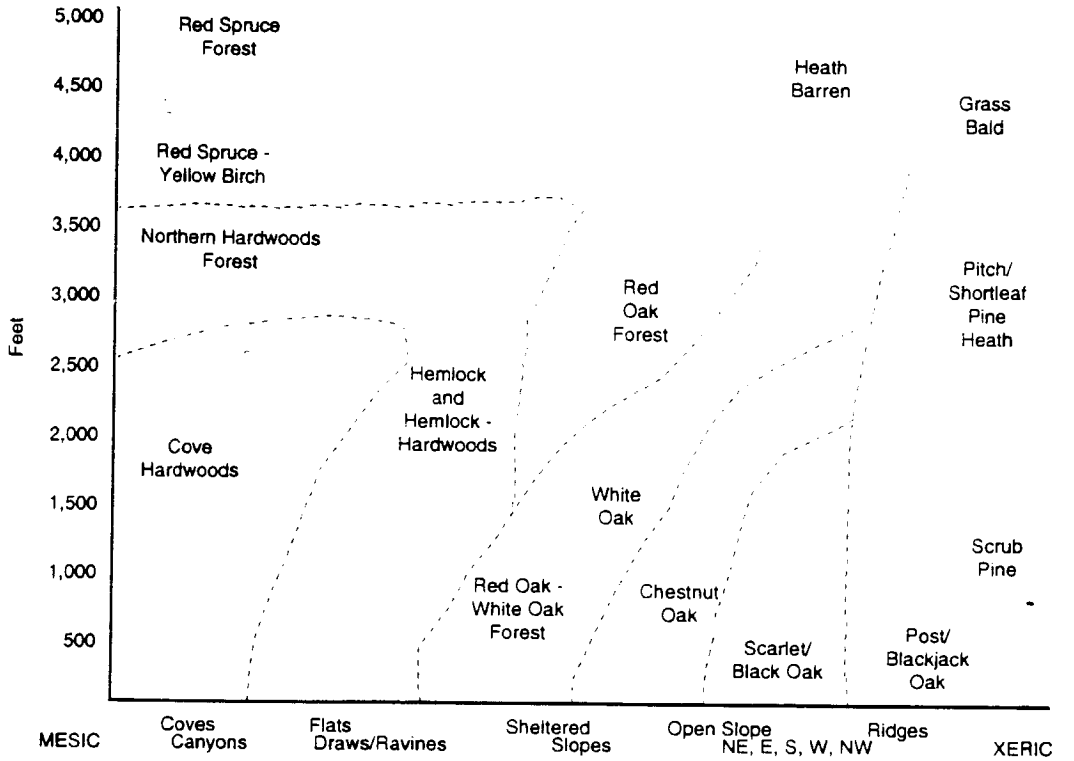

Figure 1. Environmental gradients for vegetation.

efficiency of the training data. An interagency committee, including ecologists from the Monongahela National Forest, West Virginia Division of Forestry, and geologists from the State Geologic Survey, generated a draft map of physiographic regions. They delineated sections based on geomorphology and climate. Sections were divided into subsections: those most typical of the section or those that are transitional, or irregular, to the section. Figure 3 is a draft map of these sections and subsections in West Virginia.

2. Classify stratified imagery using the ancillary data. High resolution imagery has not been used widely in the eastern United States, where forests are not homogeneous stands of relatively few tree species as they are in the West. Researchers who classified eastern forests from satellite imagery attempted to find the most distinctive spectral band combinations for discriminating cover types (10-13). One recent technique (14) uses a nonparametric approach that combines all spectral and informational categories to classify imagery. We are testing a variety of methods such as nonparametric processes, traditional clustering techniques, and use of derived vegetation indexes to find the most successful method.

3. Assess accuracy with random plots from videography. Videography will be acquired in the spring of 1995. Aerial transects, which extend the length of the state, will be flown with approximately 30-kilometer spacing. By regulating flight altitudes, the resolution per frame can be captured at 1 kilometer per frame. About 7,000 frames will be collected, which make up a 3 percent sample of the state. About 2 to 3 percent of the videography frames will be field verified. With this strategy, we will test the effectiveness of using videography, instead of intensive field plots, to verify classification of satellite imagery. Areas of special interest, which the systematic transects may not capture, will require separate flights. The bulk of the videography will provide training data for supervised image classification. The remaining frames will be used to assess the accuracy of the classification.

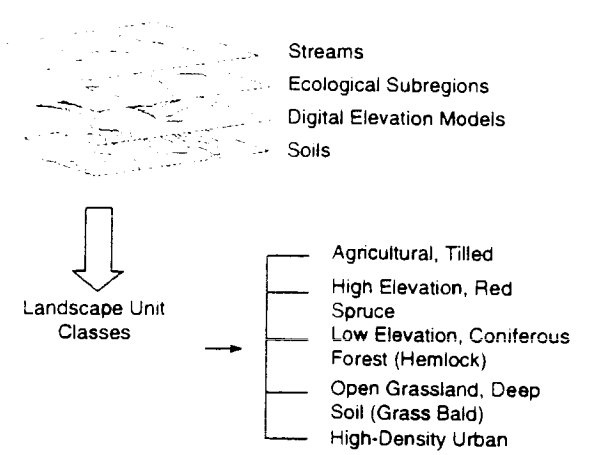

Figure 2. GIS and the development of ecological land units.

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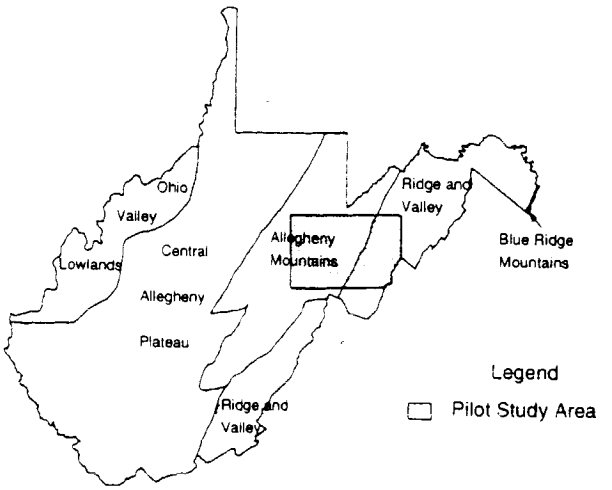

Figure 3. Physiographic regions of West Virginia and pilot study area.

In summary, 100 percent of the state will be classified using the Thematic Mapper imagery. Aerial videography, covering approximately 3 percent of the state, will help to verify the image classification, and 2 to 3 percent of the videography will be verified from transects on the ground.

4. Determine sources of data for image classification. Due to the increasing interest in GIS, digital data are more readily available from a variety of sources, such as the federal government, state agencies, and private companies. Acquisition of available data sets can substantially reduce the time and cost of database development. Users must bear in mind, however, that databases are developed with differing objectives and techniques, so one must consider scale and standards of production when deciding which data sets are appropriate for project design. The West Virginia gap team determined that the following GIS coverages are important for image classification.

The U.S. Geological Survey's (USGS's) graphic information retrieval and analysis system (GIRAS) earmarked land use/land cover data. The classification was done several years ago, and although these data are not current, they provide excellent information concerning urban and agricultural land use. Land-cover categories represent Level II classifications from Anderson's (15) system. The maps are produced at a 1:250,000 scale, so they require few GIS operations to piece together a regional coverage of land use.

The Southern Forest Experiment Station mapped U.S. forestland using advanced very high resolution radiometer (AVHRR) satellite imagery (16). This imagery is relatively current (1991 to 1992), but the resolution is coarse at 1 kilometer per pixel (100 hectares or 247 acres). The classes are based on Forest Inventory and Analysis plots established by the U.S. Department of Agriculture (USDA) Forest Service and Küchler's (17) potential natural vegetation types. We are using the maps to depict broad changes in forest type over a region, such as the state of West Virginia. This coverage does not show land use.

The eastern region of the Nature Conservancy has completed a draft of the classification of the terrestrial community alliances (18). The classification hierarchy is that prescribed by the national gap team, and as such, reflects physiognomic and floristic characteristics necessary for correlating vegetation structure and floristic composition with vertebrate habitat requirements. The descriptions include the range of alliances and characteristic species of the overstory, understory, and herbaceous layer. This provides information on associated species not detectable by image classification.

The National Wetlands Inventory data are available digitally. Maps have been digitized at a scale of 1:24,000, and the classification scheme is from Cowardin et al. (19). Coverages come with attribute data for each polygon, arc, or point as needed. A labor-intensive effort is required to join the maps in a GIS for an area the size of West Virginia, but the information will be invaluable for masking water and forested wetlands on the satellite imagery. The U.S. Fish and Wildlife Service includes detailed instructions for converting the data to coverages.

Field data, much of it already digital, has been acquired from many sources. Commercial timber companies provided data for timber stand composition and age groups. The USDA Forest Service ecologist conducted transects throughout the Monongahela National Forest to characterize ecological land units. Forest inventory and analysis plots are also available. We acquired these data to verify videography classification.

The USGS has digital data of terrain elevations. The West Virginia gap analysis team acquired 3 arc-second DEMs as an additional band in the satellite imagery. We will use these data to generate coverages of slope, aspect, and elevation classes to further stratify the physiographic regions of the state. This will increase the accuracy of the classification.

The Soil Conservation Service created a statewide database called STATSGO, produced at a scale of 1:250,000. For West Virginia, the map of the soil mapping units consists of approximately 450 polygons. Each mapping unit is an aggregation of soil components that occupy a certain percentage of the mapping unit area. The database is extensive and includes information on soil attributes such as soil taxonomy, soil chemistry, soil structure, and interpretations for erodability and wildlife habitat. For an attribute such as soil temperature, each component has an individual value so that each mapping unit may have several different values for soil

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temperature. Attribute information is difficult to query in ARC/INFO, where there is a one-to-many relationship between polygons and database entries (for instance, each mapping unit, or polygon, has several soil components). We found that exporting the attribute information from ARC/INFO to another software package such as Excel was easier. The values can be aggregated by attribute and then imported into ARC/INFO to produce individual coverages such as soil texture, soil depth, or soil group. STATSGO data can provide useful information for the physical variables that influence vegetation, such as soil moisture and nutrient availability.

To review, the project researchers will first identify the physical parameters that govern the distribution of ecological land units and the existing vegetation in the state. Then, the team will gather applicable ancillary data of physical data in GIS to support the image classification. Once the imagery has been classified, the wildlife models can be incorporated.

Integrate Terrestrial Vertebrate Models

Once classification is completed for the state, the gap analysis team will integrate terrestrial vertebrate models. Concurrent with the image classification, the team will develop a species profile for each vertebrate species known to occur in the state. These profiles, when completed, will be condensed into rule-based models for associated species that can be linked to the ecological land units (see Figure 4). This step will create habitat types. After integration, the team will generate maps that display species richness of vertebrates for each habitat type (see Figure 5). These maps link spatial data to the species database. This enables users to identify areas in the landscape that combine habitats with the greatest number of unique species. When a coverage of landownership is overlaid on this map, land managers or conservation groups can take a proactive stance to seek protection of critical habitats. Additional analyses that users can perform are displays of the potential distribution of vertebrate groups, such as upland salamanders. Another analysis would be to report the species that occur in the fewest habitats and that would be most vulnerable to landscape changes. Clearly, GIS provides a powerful environment for quick and efficient retrieval of spatial data for management decisions.

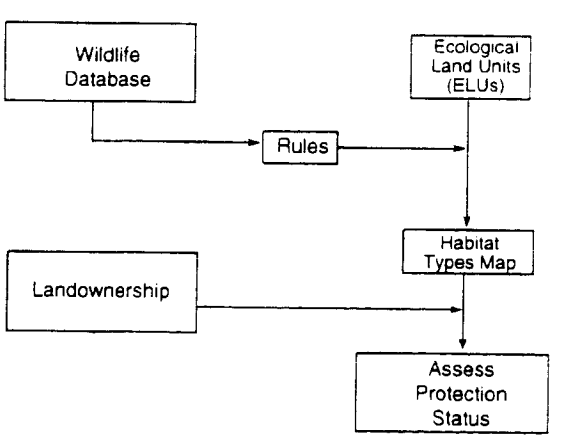

Figure 4. Informational flow chart for wildlife data.

Summary

To summarize, the West Virginia gap analysis team is assessing the natural communities in the state as a part of the national comprehensive planning effort. We need to conduct the assessment rapidly, compiling existing information and integrating these data with GIS and remotely sensed data. Ecological land units are classified according to the relationship between vegetation and ecological variables. Satellite imagery is used to map existing conditions over the state. The existing vegetation is classified to reflect physiognomic and floristic elements to correlate with vertebrate and butterfly habitat requirements.

The gap analysis team is using many widely available data sets such as DEMs, land use/land cover, wetlands inventory, and soils data. While these can reduce the time and cost of developing an ecological database, they do present implications for project design and accuracy. When the user combines maps of different scales, accuracy is constrained by the map with the smallest scale. Additionally, data sets may be constructed with objectives for an intended use that is not compatible with project needs. The classification of AVHRR data reflects forest types but not land use, so another source may be required for these data.

The Gap Analysis Project is not a substitute for intensive biological studies at a fine scale. It is merely a quick assessment at a broad scale to provide information on existing conditions. While accumulating data and modeling potential wildlife distributions, we will identify where inventory data may be lacking. Additional work must be done to verify wildlife models and the classification of vegetation, but this preliminary analysis will be a valuable framework that will direct future studies of biological diversity. Finally, this effort will provide a data set that can be used to monitor changes to land cover and land use.

Acknowledgments

This study is being funded by the Cooperative Research Units Center, National Biological Survey, United States Department of the Interior. This manuscript is published as Scientific Article No. 2499 of the West Virginia Agriculture and Forestry Experiment Station.

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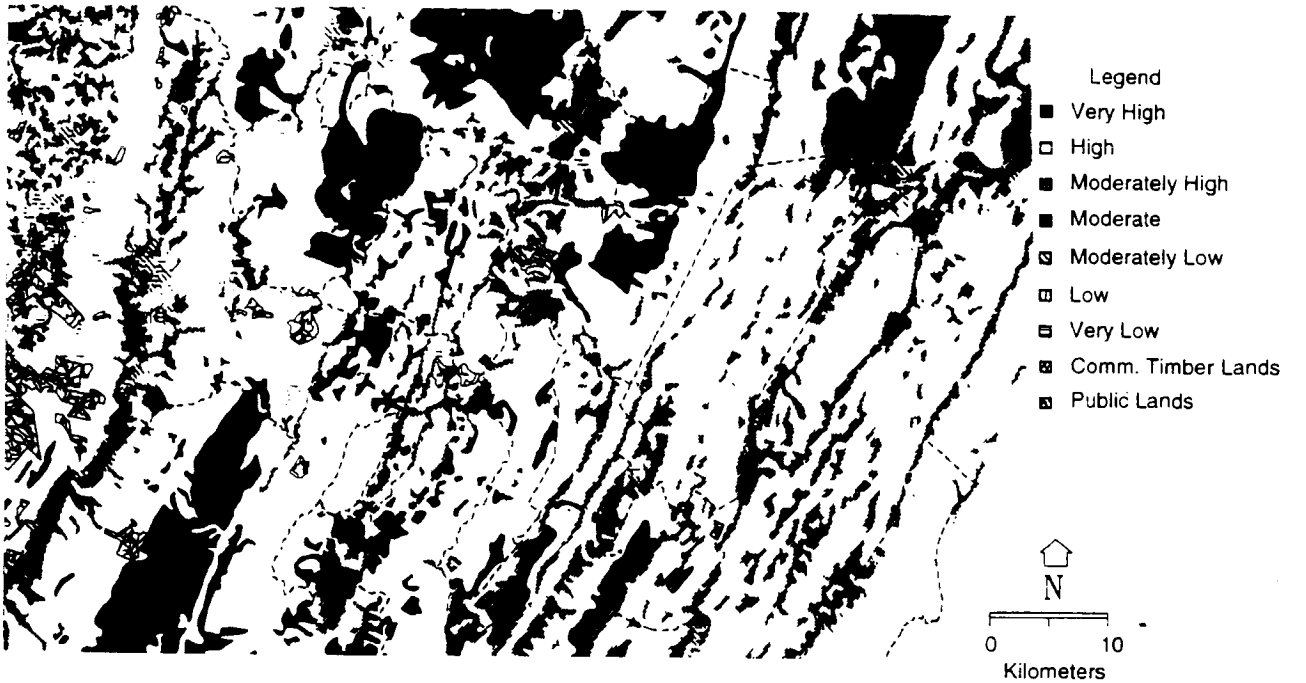

Figure 5. Relative species richness.

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A GIS Strategy for Lake Management Issues

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Abstract

Lake management plans are crucial to the sustained life of a lake as it experiences pressures from human as well as environmental activities. As proven in the past, geographic information systems (GIS) can meet the needs of most if not all environmental entities. Applying GIS to lakes and lake management, however, is a fairly new concept because most previous work focused on the terrestrial realm. Future studies must address problems relating to dimension, but adopting certain methods (i.e., cross-sectional coverages) can help lake managers plan for critical lake issues. By using sufficiently planned coverages, lake quality data management coverages can increase storage and/or analysis efficiency. After evaluating certain management criteria, a lake management plan can be derived and set up as a coverage. These criteria can then correspond collectively to form management zones within a lake. Each of these zones has its own set of management goals to which all lake users must strictly adhere.

Introduction

The importance of maintaining lake quality has long concerned recreationalists and ecologists. The multifaceted interrelationships of the lake environment, however, usually make proper assessment and analysis of lake quality information difficult. Over the last decade, assessment has become easier due to the increased use and acceptance of geographic information systems (GIS). This computer-based tool has allowed successful integration of water quality variables into a comprehensible format.

One area of the environmental sciences that has neglected GIS is lake management. This paper presents an alternative method for using a traditional two-dimensional GIS for viewing, querying, and displaying three-dimensional information—in this case, lake quality information and lake management criteria. Lakes, unlike geologic entities, offer a three-dimensional realm that humans can fully penetrate without a great amount of effort. Lakes also contain a complete aquatic environment of physical, chemical, and biological entities that humans can effectively observe and analyze. This paper does not discuss the issue of dimension; however, future studies, primarily those relating to the creation of three-dimensional GIS, should address this issue.

GIS allows incorporation of a multitude of environmental variables (e.g., water chemistry, geologic strata) into a synergism of the many coexisting variables of the lake environment. The ability of a GIS to "capture, manipulate, process, and display spatial or georeferenced data" is now well known and accepted (1). Surprisingly though, GIS is rarely used for lake management databases and associated water quality analysis.

The few examples that exist include Schoolmaster's Texas Water Development Board System (2), which examined water use on a county basis, and RAISON GIS (3), which is an expert computer system implementing proper application of hydrologic principles to a particular lake. Many other systems are simply database collection storehouses of lake information, such as the Galveston Bay National Estuary Program (4).

One notable example is the LAKEMAP program (5). This extensive and comprehensive GIS spans the entire United States covering approximately 800,000 lake sampling sites from the U.S. Environmental Protection Agency's (EPA's) STORET system. LAKEMAP uses both a database management and mapping display system, allowing retrieval of information for specific sites or aggregation of regional areas. This GIS is unique because it examined the creation of standards that could be used across the country in database development and the presentation of that data.

Using GIS in a lake management study inspires many questions because of the lack of existing research and the absence of any true standards. For example, how should one create a lake quality database for general purpose management? Is visualizing the integration of several variables within the lake ecology possible? Can

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one examine temporal changes in pH? Many technical and logistical GIS questions therefore existed when the Legend Lake study began.

Background of the Legend Lake Study

Legend Lake (see Figure 1) is located in Menominee Reservation, which is in Menominee County in northeastern Wisconsin. Legend Lake is a 1,230-acre impoundment comprising eight natural drainage lakes that a single stream once connected. In the late 1960s, a plan was introduced to convert this area into an impoundment/recreational area, and construction soon began. The ecology and hydrology had not been seriously evaluated since the development was finalized, hence the Legend Lake project was designed in cooperation with EPA, Wisconsin Department of Natural Resources (WDNR), Menominee Reservation and County, Legend Lake District, and the Legend Lake Property Owners Association. This intensive study spanned the qualitative and quantitative aspects of surface water, ground water, sediment, and aquatic plants, as well as human influences on the Legend Lake watershed and surrounding areas.

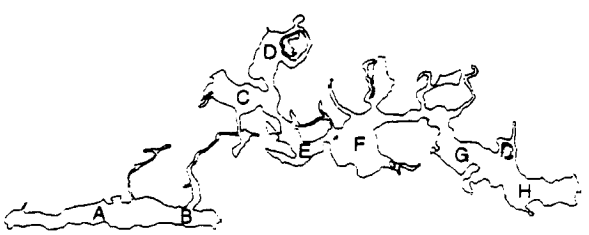

Figure 1. Legend Lake with basin identifiers.

Initially, questions needed to be answered regarding aquatic plants and sediment and their influences on lake management strategies. Because the scope of the Legend Lake study included subjects such as surface- and ground-water chemistry, land use and development, septic system impacts, and recreational stress, all these factors needed to be considered in determining optimum management strategies. In addition, the study addressed the question of GIS's ability to alleviate some technical aspects of deriving and presenting a lake management plan. Given these questions, the goal was to integrate collected data into a GIS database to create a prototype standard for future lake studies, as well as to present new techniques for visualization and analysis of lake quality data.

GIS/Database Creation

Several techniques were used to best manage the data for a three-dimensional system: cross-sectional coverages (described later), data summary coverages, and multidate coverages. The latter two coverage types are traditional coverages that contain general lake information excluding water column data. These techniques work fairly well for data storage and visualization; however, they were not sufficient for determining the geographic areas *within* the lake that required intensive management decisions as opposed to areas that needed little attention.

During formulation of a new coverage design, the study focused on the littoral zone, which is usually defined as that area of the lake with a depth of 15 feet or less. Most management concerns deal with the littoral zone because most recreational and ecological activities occur in this zone. One of the most pressing issues in lake management is aquatic plant growth. The fact that most aquatic plant growth is confined to the littoral zone reinforced the decision to use the littoral zone as the primary sink for potential management decisions.

To curtail the dimensional problem, depth was basically ignored. This allowed for easier delineation of areas within the lake. This, in turn, facilitated classifying areas into management zones to implement varying degrees of activity, ranging from casual to intensive efforts. Thus, management recommendations for a particular zone were the same at a depth of 2 feet as at a depth of 12 feet. This greatly reduced complication of the model and centered effort on the areal extent of the lake. It also facilitated visualization of management decisions by professionals and lay people.

Problems can arise when combining depths for management considerations, as this technique did. For example, a littoral zone that contains a gentle slope usually does not receive the same attention as a littoral zone with a very abrupt slope because the littoral zone with the gentle slope contains more area. Thus, if a lake manager recommends restricting boat traffic in a management zone (a section of the littoral zone) that has a gentle slope to promote wildlife habitat, the amount of area available for boaters would decrease significantly. In this situation, primary activity would be most crucial in shallow areas where wildlife or waterfowl predominate rather than in more open water areas where boaters predominate. Situations like these may require the creation of management subzones when setting up a lake management GIS and database to increase efficiency of lake area use and increase support by lake users.

Many criteria can affect the decisions made for a management zone. For instance, if the lake manager recognizes excessive, unhealthy weed growth in a particular zone, the lake manager may recommend extensive weed harvesting to neutralize the situation. An adjacent zone may have very little weed growth and may not require weed harvesting. Criteria such as these must be recognized before constructing the GIS. Table 1 lists some common criteria to consider. Generally, management criteria include anything that is influent on the

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Table 1. Criteria To Consider When Creating Lake Management Plans

Environmental Variables	Artificial Variables
Aquatic plants	Adjacent land use
Sediment	Septic systems
Ground water	Development
Surface water	Construction sites
Wildlife/waterfowl	Fuel leaks
Fisheries	Shoreland zoning
Climate	Population density
Wind	Primary lake use (recreational)
Geology/geography	Nutrient loading
Adjacent natural land cover	Visitor use
Natural nutrient loading	
Hydrologic characteristics	

shoreline and lake itself and any influence the lake has on the shoreline or adjacent shorelands.

A primary concern when accessing lake data from a computer database is being sure to query the correct lake. For example, searching a database for all data on "Sand Lake" would be a legitimate action, except that the database may include close to 200 lakes with the name of Sand Lake. This is one of the main reasons why the WDNR developed a system known as the Water Mileage System (6). Based on logical criteria, each water body (e.g., lakes, streams, sloughs) receives a unique six- or seven-digit number called the waterbody number. Thus, if the number 197900, assigned to Sand Lake near Legend Lake, is the query subject, then the output should include all data for this particular Sand Lake. Because having a unique identifier for each specific entity in a GIS database is ideal, the waterbody number was used, and all sampling performed on this lake will be linked with this number.

Examples of Types of Coverage

Management Zone Coverages

Figure 2 and Table 2 together show how a potential lake management GIS and plan might work. The lake manager can easily manage and frequently update this system if necessary, or the system can serve as a long-term plan to consult for all decision-making. A plan of this sort specifically emphasizes areas that need intensive management over areas that may need frequent monitoring. It provides specific instructions for plan implementation, leaving little guess-work for the manager. This technique is also visually informative to the lake user because the user can easily discern areas of concern. This example is hypothetical, but a plan is being formulated based on the information collected during the Legend Lake study.

Cross-Sectional Coverages

Another technique currently included in the Legend Lake study entails the z dimension. Cross-sectional views of each lake basin, derived from 1992 lake contour maps, provided a more detailed description of the lake bottom. These cross sections were then digitized and transformed into GIS coverages. For each lake basin, 22 tests were conducted on the deepest part of the lake at several different depth intervals along the water column. These data provided valuable information on the way various chemical and biological attributes react to depth. Using GIS, a point could represent each depth where data were collected. These points could actually act as labels for polygons based on depth.

For example, if performing a series of analyses on a lake (maximum depth of 10 feet) at 3-foot, 5-foot, and 8-foot depth intervals, the labels on the cross-sectional coverage would be placed at these respective depths. Thus, labels would be positioned at depths of 3, 5, and 8 feet. Because these labels represent cross-sectional polygons, the 3-foot depth label may represent a polygon with boundaries at 0 feet and 4 feet. The 5-foot depth label may represent a polygon with boundaries at 4 feet and 6 feet, and the 8-foot depth label may represent a polygon with boundaries at 6 feet and the lake bottom (10 feet). Those who are familiar with lake ecology understand that no clear-cut boundaries distinguish where chemical values jump from one measurement to another without a gradual transition. All users of a lake quality GIS must be made aware of these types of inaccuracies (see Figure 3).

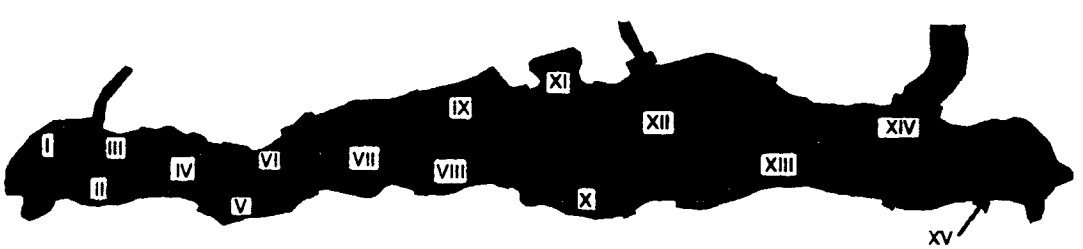

Figure 2. Hypothetical management zones for a section of Legend Lake that correspond to the management plans in Table 2; black areas indicate depths greater than 15 feet.

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Table 2. Hypothetical Management Plans for a Section of Legend Lake (see Figure 2)

Management Zone	Management Plan (brief explanations)
I	High recreation area, high plant growth, frequent harvesting; frequently monitor water quality
II	Moderate recreation; manage for fish habitat
III	Moderate recreation; manage for fish habitat
IV	Open water, high recreation/possible fish habitat; consider subzoning
V	High-grade wildlife habitat; restrict human contact
VI	Moderate recreation; manage for fish habitat
VII	Open water, high recreation, increased shoreland development; frequently monitor water quality
VIII	Adjacent to high recreation area, possible fish habitat; manage for fish and aquatic habitat
IX	Adjacent to high recreation area, shoreland development; monitor water quality
X	Increased development; frequently monitor water quality
XI	[a]Prime wildlife/waterfowl habitat adjacent to high recreation area; restrict human presence (hot spot)
XII	Open water, high recreation area; frequently monitor water quality
XIII	Open water, high recreation area, possible fisheries and wildlife habitat; consider subzoning
XIV	Excessive aquatic plant growth (species listed) choking out preferred species; potential wildlife/waterfowl habitat, fisheries potential; continual harvesting; restrict human presence
XV	High-grade slope with little plant growth, potential for increased sedimentation; monitor shoreland development

[a]Critical area between good habitat and high recreation area; monitor extensively.

Figure 3 shows a cross section from one of the larger basins, Basin F, in the Legend Lake system (see Figure 1). The shades of gray represent ranges of temperature, with the lightest being the coldest and the darkest being the warmest. The thermocline can be located roughly in the middle. Each colored section represents a depth range where certain chemical attributes were collected. Using ARC/VIEW, the user can choose these areas with a pointer (mouse) and gain access to the database that contains all the sample results for this depth range.

Conclusion

These ideas are still preliminary as the Legend Lake study analysis concludes. Clear-cut discussions and recommendations will become available at a later date, although some observations can be made at this point. First, future research should focus on creating and developing a three-dimensional GIS, not to be confused with three-dimensional or cartographic models. Ideally, lake management plans should consider depth. This paper did not include depth because of the dimensional factor. Depth was not compatible with our two-dimensional GIS, and the presentation quality was not sufficient to relay our results in the form of management zones. We must develop a three-dimensional GIS to address the problems of the three-dimensional environment in which we live.

Lastly, maps are the best form for communicating this information to professionals and the public. Maps can also easily confuse people, however. Proper cartographic techniques are a necessity (7). Significant effort must be devoted to map creation to ensure a successful plan and successful relationships between lake managers and lake users. GIS and map-making are closely related. Both the planning stages and the database development phase of the lake quality GIS should emphasize this point. At an early stage of the process, management criteria should be determined, and all players or potential players must be included. A poorly planned project can lead to a failed GIS.

Creativity may offer new ideas in map development. For instance, animation (8) has some unique traits. Trend analysis using animation may produce the best visual results. Techniques such as these augment our methods of communication, and some are very revolutionary. Remember, however, that cartographic principles still must apply.

Acknowledgments

I would like to thank Dr. Keith Rice of the University of Wisconsin, Stevens Point, for his review efforts as well as Dr. Byron Shaw and Steven Weber for their technical support on lake management issues. Also, I would like

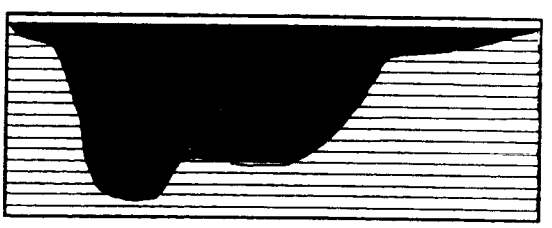

Figure 3. Cross section from one of the larger basins, Basin F, in the Legend Lake system.

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to thank members of the University of Wisconsin, Stevens Point, Geography/Geology Department for their support and for the use of their equipment.

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A Watershed-Oriented Database for Regional Cumulative Impact Assessment and Land Use Planning

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Introduction

In 1974, North Carolina passed the Coastal Area Management Act (CAMA) to guide growth and development in the state's coastal zone. Today, the Division of Coastal Management (DCM), under the direction of the governor-appointed Coastal Resources Council, implements CAMA. DCM's jurisdiction covers the 20 counties that border either the Atlantic Ocean or the Albemarle-Pamlico estuary.

This coastal region comprises a diverse set of human, animal, and plant communities. A broad array of coastal plain ecosystems occurs in this area, from the barrier dunes and maritime forests of the outer banks to cedar swamps and large pocosin complexes of interior areas. This area includes some of the state's fastest growing counties and some that are losing population. Urban centers such as Wilmington do exist, but the region remains primarily rural.

In recognition of the 20th anniversary of the passage of CAMA, the governor designated 1994 as the "Year of the Coast." Associated celebrations, panels, and studies highlighted the unique features of the North Carolina coast, successes of coastal management in the state, and unresolved problems and concerns. Problems remain despite protection efforts by various agencies. For instance:

- Fish landings have dropped dramatically of late.
- Shellfish Sanitation recently closed a set of shellfish beds located in outstanding resource waters.
- Shellfish statistics show that the quality of the state's most productive coastal waters continues to decline.

Because coastal North Carolina as a whole is growing more rapidly than any other section of the state, pressures on coastal resources can only continue to increase.

Declining water quality and associated sensitive habitats, resources, and animal populations have prompted several state agencies to develop new approaches to environmental protection that incorporate a broader, natural systems perspective. The North Carolina Division of Environmental Management is developing river basin plans to guide point and nonpoint water pollution control efforts. The DCM has begun work to assess and manage the cumulative and secondary impacts of development and other land-based activities by using coastal watersheds as the basis for analysis. The goal of this work is to expand the regulatory and planning programs in order to better address cumulative impacts. This paper describes the approach that DCM has developed for cumulative impacts management, with special emphasis on the use of a geographic information system (GIS). The project described here is scheduled for completion by the fall of 1996.

Cumulative Impacts Management

The concept of cumulative impacts management (1) is not new to North Carolina's coastal program. CAMA requires the consideration of cumulative impacts when evaluating development permits within defined areas of environmental concern. A permit must be denied if "the proposed development would contribute to cumulative effects that would be inconsistent with the guidelines. . . ." Cumulative effects are defined as "impacts attributable to the collective effects of a number of projects and include the effects of additional projects similar to the requested permit in areas available for development in the vicinity" (2). Despite this directive, few permitting actions have been denied because of cumulative effects; the existence of limited impact data and a dearth of viable analysis approaches have restricted application of this rule.

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Since the passage of the National Environmental Policy Act (NEPA) in 1969, many attempts have been made to define and assess cumulative impacts. The Council on Environmental Quality developed the most familiar definition in its guidelines for NEPA implementation. It defines cumulative impact as:

> . . . the impact on the environment which results from the incremental impact of the action when added to other past, present, and reasonably foreseeable future actions, regardless of what agency or person undertakes such other actions. Cumulative impacts can result from individually minor but collectively significant actions taking place over a period of time (3).

Although this definition focuses the discussion of cumulative impacts, it provides little guidance on how to carry out such an analysis. Selecting both an appropriate time frame for the assessment (how far into the past and future to carry the analysis) and appropriate boundaries for the study (municipal or county boundaries, watersheds, ecoregions) are but two of the questions that require answers to successfully investigate cumulative impacts. Such decisions become even more complex when incorporating the limits imposed by available data and existing management structures.

Rigorous cumulative impact analysis is a difficult proposition. It requires identification of all sources of degradation that affect a given resource. The next step involves assigning relative significance to each of these sources along with any impacts that result from additive or synergistic interaction between sources. Assessment of the impacts of a pier on surrounding sea grasses, for instance, must include not only impacts related to the structure, such as shading and wave or current changes, but also such ambient impacts as natural wave and wind effects, upland runoff, and varying salinity.

All these investigations require the availability or collection of baseline environmental data at an appropriate spatial and temporal scale. Quantifying all the sources and causal pathways that affect a resource is extremely complicated in all but the simplest of systems. Assigning proof of significant impact is difficult unless the cause is clear and direct.

Because of the difficulties associated with assigning cause in cumulative impact analysis, especially in a regional review, DCM has chosen a different approach. It is focusing instead on locating areas at high risk to cumulative impacts. Impacts management studies and responses can then target the areas at greatest risk of degradation. Changing the scale of analysis from the site to the region requires applying some simplifying assumptions. The first assumes that any existing resource degradation results from the cumulative impact of all sources within the system boundaries. Locating such areas is relatively straightforward because most natural resource fields have developed measurements and indicators for locating degraded resources. The second assumption claims that a sufficiently intensive concentration of activities within a limited area will result in cumulative impacts on the affected system. Determining a threshold beyond which impacts cause degradation is much harder than locating already degraded resources because the level of such a threshold depends upon both the strength or spatial concentration of the impacts and the sensitivity of the resource.

Working from these simplifying assumptions, DCM's first step in assessing regional cumulative impacts is to identify areas within coastal North Carolina that exhibit symptoms of resource degradation, contain a concentration of activities that affect resources, or contain a concentration of sensitive resources. The use of categories of resources and impacts have helped to focus this search. These eight cumulative impact, high-risk area categories are:

- Impaired water quality
- High potential for water quality impairment
- Sensitive ground-water resources
- Impaired air quality (present or potential)
- Historical rapid growth
- Anticipated high growth
- High-value resources
- Productive and aesthetic resources

This set of categories is presently under public review. The next step is to develop indicators of the presence of impacts or resources appropriate to each of these categories. These indicators, when applied to a database of information about the study area, will help identify those locations at high risk as defined by the eight categories.

The regional cumulative impacts assessment approach that DCM developed is a hybrid of various assessment techniques. The overall approach is grounded in the theory and methods of site-specific cumulative impact assessment. Determination of high-risk categories and appropriate indicators and indexes is closely associated with both relative risk assessment procedures and geographic targeting. By focusing on known causes and effects of cumulative impacts on terrestrial and aquatic natural resources instead of attempting to quantify all impact pathways, available data and analysis techniques can help assess relative risk of cumulative impacts.

A Watershed Database for Cumulative Impact Assessment

High-risk categories and indicators of degradation or sensitivity are useless without information on the location

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of sensitive resources and impact sites. Consequently, a comprehensive database of information about coastal North Carolina is central to cumulative impacts management in this area. The form of any database determines what types of questions to ask it; the selection of boundaries has been central to this study.

County boundaries constitute the most typical reporting unit in DCM operations. Counties determine the boundaries of DCM's jurisdiction, and the great majority of statistics used in planning and assessment are available primarily or solely by county. County size, heterogeneity, and the small number of counties available for comparison, however, have made county boundaries inappropriate for this project. Because the study focuses on impacts on natural resources, clearly the most appropriate boundaries would relate more directly to those resources.

Although using a single set of boundaries may not be appropriate for assessing impacts on all resources, management constraints limit the choice to one boundary type. Because the primary resources of concern are water based, watersheds were considered most appropriate. Surface waters receive the integrated effects of activities within a watershed; such boundaries fit intuitively with the concept of cumulative impact assessment. The number of water-related resources of concern also supported this choice. This analysis used small watersheds (5,000 to 50,000 acres) delineated in 1993 by the Soil Conservation Service for the entire state of North Carolina.

The Population, Development, and Resources Information System (PDRIS), which was designed for this project, is a PC-based, watershed-oriented database that contains the following information about the coastal area:

- Natural resources
- Population and housing
- Agricultural activities
- Economic activities
- Development activities

Table 1 includes a list of database fields. The presence and extent (or absence) of each of the features that this database represents will be available for each coastal watershed. The small watershed orientation of this study is only possible because of the availability of GIS; the volume and complexity of the watershed boundaries preclude any other assessment tool. In fact, 348 of these watersheds fall wholly or partially in the 20-county region. Figure 1 shows a map of these small watersheds. This map indicates county boundaries and shorelines in solid lines and the watershed boundaries in gray.

Data Needs and GIS Analysis

Over the past 5 years, North Carolina has actively collected a large amount of natural resource and base map information in GIS form. Research and funding associated with the Albemarle/Pamlico Estuarine Study (APES), a national estuary program study, spurred much of this data development in the coastal area. The state maintains a central repository for geographic data at the North Carolina Center for Geographic Information and Analysis (CGIA). Table 2 lists the general types of information available from the state database. The availability of data in GIS form is but one criterion for selecting a data set for use in this analysis. To be useful, the scale and accuracy of the data must be appropriate to the analysis.

Data Scale

The majority of data in the state's GIS database was collected at a scale of 1:100,000. Broader use and interest will probably urge the development of data layers at finer scales. A recently released layer of closed shellfish waters, for instance, was created at 1:24,000 scale. This prompted an update of the associated shoreline coverage to the same base scale. A handful of state departments and divisions, including Coastal Management, now use global positioning systems to collect even more precise locational information. This scale suits DCM's regional cumulative impacts scan, which is based on summary values for entire watersheds. More detailed intrawatershed planning and analysis would require finer scale data. A scale of 1:24,000 delineated the watershed boundaries in this project.

Mixing these 1:24,000 boundaries with the 1:100,000 data sets, however, can cause problems. For instance, a number of watersheds were designated for the large open water areas in Albemarle and Pamlico sounds. Although these should comprise exclusively water, overlay analysis of these watershed boundaries on the TIGER-derived census boundaries (1:100,000 scale) resulted in the assignment of small population counts to some of these watersheds. Individually locating and correcting such discrepancies is necessary.

Database Accuracy

Data layer accuracy problems are difficult to identify and assess. Because other agencies developed the majority of data used in this project, these source agencies must be relied upon for accuracy assessment of the source data. CGIA, steward of the state GIS database, adheres to National Map Accuracy Standards for all GIS data that it maintains. CGIA delivers metadata reports with any data; these reports include the source agency, collection date, and scale for the information used to derive the GIS layer. Descriptions of data lineage (collection and processing procedures), completeness, and positional

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Table 1. Population, Development, and Resource Information System: Database Fields

Agriculture: Livestock and Poultry
Beef feedlots (< 300 head, > 300 head)
Dairy farms (< 70 head, > 70 head)
Hog farms (< 200 head, > 200 head)
Horse stables (< 200 head, > 200 head)
Poultry farms (< 15,000 birds, > 15,000 birds)

Agriculture: Farming
Land in farms (acres, % of HU)
Land with best mgmt. practices (acres, % of HU)
Land w/o best mgmt. practices (acres, % of HU)
Land in conservation tillage (acres, % of HU)
Land w/o conservation tillage (acres, % of HU)
Harvested cropland (acres, % of HU)
Hay crops (acres, % of HU)
Irrigated land (acres, % of HU)
Pasture land (acres, % of HU)
Row crops (acres, % of HU)

Primary
Estuarine waters (acres, % of HU)
Freshwater lakes
HU name
Receiving HU
Receiving water body
Primary water body
Secondary water body
Shoreline
Waterways w/vegetated buffers (miles, % of HU)
Population 1970
Population 1980
Population 1990
Population growth 1970 to 1980
Population growth 1980 to 1990
Counties
Total HU size
Land area (acres, % of HU)
Water area (acres, % of HU)
Stream length (miles)
Stream order (miles, % of stream length)

Development
Building permits—all residential
Building permits—amusement/recreation
Building permits—multifamily residential
Building permits—one-family residential
Building permits—hotels and motels
Building permits—retail
Building permits—industrial
Highway mileage:
- Total (miles)
- Primary (miles, % of total)
- Secondary (miles, % of total)
- Paved (miles, % of total)
- Unpaved (miles, % of total)

Rail lines (miles)
Increase of primary & secondary roads (miles, %)
Increase of paved vs. unpaved roads (miles, %)

Economic
Ag-related business (number, employees, income)
Farms (number, employees, income)
Fisheries business (number, employees, income)
Forestry/wood-using business (number, employees, income)
Lodging establishments (number, employees, income)
Manufacturing establishments (number, employees, income)
Marinas (number, employees, income)
Mining establishments (number, employees, income)
Recreation business (number, employees, income)
Restaurants (number, employees, income)
Retail establishments (number, employees, income)

Ground Water
Ground-water contamination incidents
Ground-water class (acres, % of HU)
Ground-water contamination area (acres, % of HU)
Ground-water capacity use areas (acres, % of HU)

Land and Estuarine Resources
Anadromous fish streams (miles, % of streams)
Coastal reserve waters (acres, % of HU)
Coastal reserve lands (acres, % of HU)
Federal ownership:
- National parks (acres, % of HU)
- National forests (acres, % of HU)
- Military reservations (acres, % of HU)
- USFWS refuges (acres, % of HU)
- Federal ownership—other (acres, % of HU)

State ownership:
- Game lands (acres, % of HU)
- State parks (acres, % of HU)
- State forests (acres, % of HU)
- State ownership—other (acres, % of HU)

Natural heritage inventory sites (count)
Primary nursery areas (acres, % of water area)
Private preservation (acres, % of HU)
Secondary nursery areas (acres, % of water area)
Threatened/endangered species habitat
Water supply watersheds (acres, % of HU)

Land Use
Total wetland area (acres, % of HU)
High-value wetlands (acres, % of HU)
Medium-value wetlands (acres, % of HU)
Low-value wetlands (acres, % of HU)
Predominant land cover

Population and Housing
Average seasonal population
Peak seasonal population
Units without indoor plumbing
Units with septic tanks
Units on central water systems
Units on central sewer
Units with wells

Permits
Air emission permits—PSD
Air emission permits—toxic
CAMA minor permits
CAMA general permits
CAMA major permits
CAMA exemptions
CWA Sect. 404/10 permits
Landfill permits—municipal
Landfill permits—industrial
Nondischarge permits
NPDES permits—industrial
NPDES permits—other
NPDES permits—POTW
Stormwater discharge permits
Sedimentation control plans
Septic tank permits

Shellfish
Shellfish waters (acres, % of HU)
Shellfish closures—permanent (acres, % of HU)
Shellfish closures—temporary (acres, % of HU)

Water Quality—Open Water
Class B waters (acres, % of water area)
Class C waters (acres, % of water area)
HQW waters (acres, % of water area)
NSW waters (acres, % of water area)
ORW waters (acres, % of water area)
Swamp waters (acres, % of water area)
SA waters (acres, % of water area)
SB waters (acres, % of water area)
SC waters (acres, % of water area)
WS-I waters (acres, % of water area)
WS-II waters (acres, % of water area)
WS-III waters (acres, % of water area)

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Table 1. Population, Development, and Resource Information System: Database Fields (Continued)

Water Quality—Streams
Class B streams (miles, % of streams)
Class C streams (miles, % of streams)
HQW streams (miles, % of streams)
NSW streams (miles, % of streams)
ORW streams (miles, % of streams)
Swamp water streams (miles, % of streams)
SA streams (miles, % of streams)
SB streams (miles, % of streams)
SC streams (miles, % of streams)
WS-I streams (miles, % of streams)
WS-II streams (miles, % of streams)
WS-III streams (miles, % of streams)

Key
HU = hydrologic unit
PSD = point source discharges
POTW = publicly owned treatment work
NPDES = National Pollutant Discharge Elimination System
HQW = high-quality waters
NSW = nutrient-sensitive waters
ORW = outstanding resource waters
SA = saltwater classification A
SB = saltwater classification B
SC = saltwater classification C
WS1 = water supply classification 1
WS2: water supply classification 2
WS3: water supply classification 3

Water Quality—Use Support
Algal blooms (count, extent/severity)
Fish kills (count, extent/severity)
Streams fully supporting (miles, % of streams)
Streams support threatened (miles, % of streams)
Streams partially supporting (miles, % of streams)
Streams nonsupporting (miles, % of streams)
Waters fully supporting (acres, % of water area)
Waters support threatened (acres, % of water area)
Water partially supporting (acres, % of water area)
Waters nonsupporting (acres, % of water area)

accuracy are not available from these standard metadata reports, however.

DCM's cumulative impacts analysis also incorporates information not available from the state GIS database. Some of this information, such as business locations, is available from private data providers. Other information, especially agricultural statistics, does not presently exist in GIS form. Non-GIS formats include county statistics, voluntary compliance databases with self-reported coordinates, and other tabular databases. Typically, little quality control has been performed on any coordinate information. When the data originate from other state agencies, DCM is often the first user of the data outside of the source agency.

GIS Analysis Procedures

This study involves no sophisticated GIS analysis procedures. GIS helps to generate summary statistics by watershed for each of the database features. GIS drawing and query operations allow analysis of database accuracy. If the data are acceptable, the next step requires overlaying the watershed boundaries on the feature and assigning the appropriate watershed codes to all features that fall within the study area. Statistics can then be generated on the number of points, length of lines, acreage of polygons, or a total of any other numeric field in the feature coverage. Finally, the resulting summary file is converted to the format that PDRIS requires. A macro has been developed to complete this analysis. This macro generates a page-size reference map, performs the overlay, generates the watershed summary statistics, and converts the statistics to the final PC format.

Extra steps are necessary to analyze any information that does not already exist as a GIS coverage. Typically, these are tabular summaries associated with a specific boundary layer, such as county or U.S. Census statistics. These cases entail overlaying the watershed boundaries on the reporting unit boundaries; the data are distributed to the watershed in direct proportion to the percentage of the unit that falls into the watershed.

For instance, if a census tract falls 30 percent into watershed A and 70 percent into watershed B, 30 percent of the total tract population will be assigned to watershed A and the remainder to B. After performing all assignments, summary statistics are again generated by watershed. This procedure assumes that the distribution of the feature is even across each reporting unit. Rarely is this a valid assumption, but when the units are considerably smaller than the watersheds, as is the case with census tracts and blocks, this assumption introduces only limited errors. Watershed estimates based solely on county statistics, however, can be grossly inaccurate. When working with county information, therefore, using covariate information that ties more precisely to specific locations is necessary. Cropland location derived from the LANDSAT land cover layer, for instance, can be used to better distribute county-level agricultural statistics.

Database and Analysis Documentation

Data documentation is essential to this project. Given the large number of fields in the final database and the

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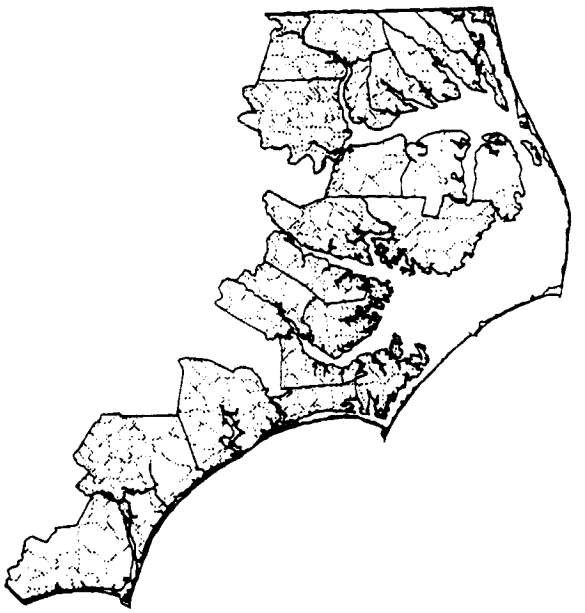

Figure 1. Watersheds in the North Carolina coastal area.

correspondingly large number of data types and sources, such documentation is key to understanding the quality of the individual database components as well as easing future database additions and updates. Because the results of this cumulative impact analysis exercise will be used to extend DCM's resource management efforts, documenting data sources and analyses will be critical if any decisions made based on this information are disputed.

A metadata database has been developed to document PDRIS data sources and analysis procedures. For each database entry, fields exist for a description and contact, collection methodology, and geographic extent of the source data. Data selection, overlay, and conversion procedures are also documented, along with any assumptions made in the analysis. In addition, recording data source, analysis procedure, and the date facilitates future database updates. Fields also record accuracy assessments for positional and attribute accuracy, logical consistency, and completeness.

The restrictions listed above regarding source data accuracy assessment, however, have limited their use. Once an entry is made to the PDRIS, all project team members receive metadata reports along with a reference map for a final review of completeness of the source data, data selection, and analysis logic. Figure 2 shows an example of a blank metadata worksheet.

Status of the Cumulative Impacts Assessment

DCM is presently gathering, verifying, and analyzing information for entry into the PDRIS. Although each source data layer was checked for accuracy before use, the logical consistency of each of the database entries relative to the other components also needs addressing. One example of such database inconsistencies is the watersheds that are covered entirely by water but also, according to the database, support a resident population. These inconsistencies could result from problems related to scale, differing category definitions, data inaccuracies, or errors in the GIS conversion or analysis at DCM. Database precision is essential for an accurate analysis and for general support of DCM's cumulative

Table 2. A Sample of North Carolina GIS Database Contents

Type	Examples	Coverage
Natural resource	Fishery nursery areas	Coastal North Carolina
	Natural heritage sites	Statewide
	Detailed soils	Varied
	Closed shellfish areas	Coastal North Carolina
	Water quality use classes	Statewide
	Detailed wetlands maps	Varied
Base data	Hydrography (24K, 100K)	Statewide
	Roads/transportation	Statewide
	County and city boundaries	Statewide
	LANDSAT-derived land cover	Coastal North Carolina
Ownership	Federal and state ownership	Statewide
Permits, waste sites	NPDES permit site	Statewide
	Landfills, hazardous waste, Superfund	Statewide
Cultural, population	TIGER boundaries, census information	Statewide
	Historic register sites, districts	Statewide

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General Information:					
Field					
Description					
Database					
Definition	0	0		Units	
Source Data Description:					
Contact					
Data					
Scale					
Sample Method					
Geographic Extent					
Database Entry:					
Procedures					
Assumptions					
Accuracy Assessment:					
	Overall	**Positional**	**Attribute**	**Logical Consistency**	**Completeness**
Rating					
Logic Test					
Error Comment					
Value Range					
Update Procedure:					
Next Update					
Procedure			Source		

Figure 2. Population/Development database: Data dictionary

impacts approach. Because the watershed database produced for this project will be widely available, errors and inconsistencies will undermine support for the rest of the project. Careful documentation of data sources, limitations, and analysis assumptions and procedures will provide useful support should problems or concerns arise.

Once database development is sufficiently complete (the database encompasses much dynamic data and could be constantly updated), indexes describing each of the cumulative impact, high-risk areas must be finalized. Applying these indexes to the database will allow identification of the watersheds at highest risk to cumulative impacts. Discussions held concurrently with index development will determine which management responses are appropriate to each high-risk category. Possibilities include strengthened land use planning requirements, new permit standards, or the designation of a new type of environmental critical area.

Although the data-intensive approach that DCM has chosen relies heavily on a GIS, the greatest challenges in this project do not lie in the GIS analysis. Applying this watershed-based analysis to existing political jurisdictions will be a more difficult undertaking. A convincing demonstration of the importance of including a natural systems perspective into a development permitting system, land use plan, or even economic development strategy, will ultimately contribute more to environmental protection in coastal North Carolina than any individual regulation that emanates from this project.

Summary

Twenty years after the passage of CAMA, DCM has developed a framework for a consistent approach to the problem of cumulative impacts of development. The approach and PDRIS database combine existing natural resource management techniques to locate areas of the coast at greatest risk of serious impairment from cumulative impacts. The availability of natural resource data at an acceptable scale (1:100,000) eases the development of the database essential to this analysis. The simultaneous development of a set of comprehensive small watershed boundaries for the state, along with the initial planning of this project, provided the final critical component to DCM's approach.

Perhaps more importantly, both DCM and individual local governments will have a large volume of information on natural units, which will provide an important, new perspective on the problems and prospects for local governmental action.

This project will not solve all problems related to cumulative impacts. The PDRIS will provide little support for site-specific or within-watershed cumulative impacts analysis; such an analysis at fine scales requires a much more precise database. By providing a broader-scale framework for this discussion, however, DCM's regional cumulative impacts study will hopefully further discussion, understanding, and management of cumulative and secondary impacts on natural systems.

References

1. Wuenscher, J. 1994. Managing cumulative impacts in the North Carolina coastal area. Report of the Strategic Plan for Improving Coastal Management in North Carolina. North Carolina Division of Coastal Management.
2. North Carolina General Statutes (NCGS) 113A-120.
3. 40 CFR §1508.7.

R0021890

A GIS Demonstration for Greenbelt Land Use Analysis

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Purpose

The goal of this project was to demonstrate what analyses could be undertaken with a GIS program without substantial GIS training or time input. The demonstration attempted to show how planning staff and decision-makers could easily and usefully employ GIS. It was not intended as a complete study of all possible variables. Only available data were used. Diverse techniques were presented while keeping the content as simple and relevant as possible. The project was designed as a demonstration using regional scale data and was combined with another parcel-specific demonstration that showed urban GIS applications.

The demonstration showed the following modeling techniques:

- Buffer zones
- Combination of variables (overlays)
- Weighting of values
- Absolute value variables
- Reclassification of final values

Site Location

The San Luis Obispo watershed comprises an area of approximately 84 square miles. The watershed drains into the Pacific Ocean at Avila, California. The major creek in the watershed, San Luis Obispo Creek, is a perennial creek, but many of its tributaries have only seasonal flow. Agriculture and grazing are the major land uses in the watershed, although a significant number of areas have been developed. Growth of these areas is moderate to limited but has a pronounced effect on the watershed. The watershed also supports a large amount of riparian and other natural vegetation. Figure 1 demonstrates the distribution of land cover/land use within the watershed.

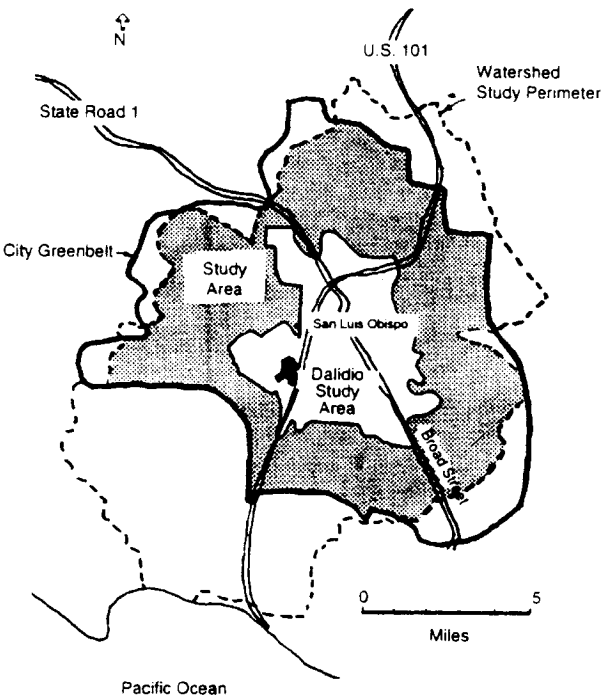

Figure 1. A generalized map of the San Luis Obispo area showing the location of the two ARC/INFO GIS study areas: (1) the parcel analysis in the Dalideo area and (2) the regional analysis in the city greenbelt stippled area surrounding the city. All boundaries are approximate and are for schematic purposes only (1, 2).

Background

The City Council approved the open space element of the San Luis Obispo General Plan in January 1994 and identified a greenbelt area that extended from the Urban Reserve Line approximately to the boundaries of the San Luis Obispo Creek watershed. The intent of this greenbelt area is to provide a buffer to the city and to preserve the agricultural and natural resources of the area.

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The data for the watershed were already available through the work of the Landscape Architecture Department of California Polytechnic State University at San Luis Obispo for a study of San Luis Obispo Creek (1), and the city's greenbelt area lay approximately within those boundaries (see Figure 1). Variables were selected that could be extracted from the available data.

The data for the creek study were initially entered into workstation ARC/INFO in a polygon format. They were then transferred to MacGIS, a PC raster program, for simplicity of use. The final product was then transferred back to ARC/INFO as a grid format and viewed in a PC version of ARC/VIEW using DOS files.

In interpreting the overlay of values, the assumption was made that the occurrence of high values for the most variables would result in the most suitable land for that land use. This was presented as a range of values derived from the total values divided into three approximately equal groups of high, medium, and low. In addition to providing a composite analysis, however, any one of the data sets can be queried separately such that, for example, slopes greater than 20 percent could be identified or two layers such as storie index and distance from roads could be compared.

Criteria

Note that the ratings of high, medium, and low are based on available data, and the rating of low implies no suitable use. In addition, these ratings do not imply that categories rated low could not be used for a particular land use but rather that other land uses might be more appropriate. For example, open space use was rated low for flatter slopes but only because this category would likely be more suitable for agriculture.

The demonstration used an existing cell size of the data on the MacGIS program of 75 meters per side, which is assigned during the initial conversion process. Therefore, buffer zones are in aggregates of 75 meters. This size cell does not allow for minute analysis but reduces the size of the files, which may become extensive in raster format.

In presenting the final analysis, land contained within the urban reserve limit line has been excluded.

Procedure

Initially, eight variables from the available data were deemed suitable for this analysis:

- Slope
- Storie index (indicating soil fertility)
- Distance from major roads
- Distance from creeks
- Erosion hazard[1]
- Oak woodlands
- Land use compatibility
- Grasslands[2]

After selecting the six variables, the categories were recoded to conform to a rating of high, medium, or low.

Each land use was then evaluated separately for every variable except for combining the variables of slope and distance from creeks for rangeland analysis. In this case, composite values were assigned to the two variables, then recoded to produce a high, medium, or low rating.

After obtaining maps for each of the variables according to land use, the maps were compiled to indicate the density of overlays for each land use category. In assessing the suitability of land for the three land uses, the values of all the variables, except land use, were aggregated and a rating system developed. In addition, a double weight was assigned to the storie index in evaluating agriculture (because this is a primary index for considering prime agricultural land). If less than 75 meters, the distance from creeks also received added weight in consideration of open space preservation (because this is likely to ensure the least erosion and pollution to the waterways). The weighting then altered the scores as follows:

Land Use	Attributes	Number of Values
Agriculture	5	6
Rangeland	4	4
Open space	5	6

After the values had been assigned for each ranking, further recoding established three categories of high, medium, and low for each land use.

The land use buffer was added to this recoded aggregate map and resulted in an additional three values due to the interaction of the buffer with each category. These additional values were recoded according to each land use to produce a final map with three values.

The Urban Reserve Area was overlaid on the final map to exclude urban areas.

Assumptions

In determining what properties would be most suitable for each land use, the following assumptions were made.

[1] After reviewing the material, erosion hazard was eliminated because it was similar to the Soil Conservation Service (SCS) storie index data while the identification of native grasslands fell only within the area currently designated as open space land, so it was not included.

[2] See above note.

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Open Space

This land is desirable to preserve as open space because of the existence of scenic or significant natural resources. It could also be land that is inappropriate for other uses due to the presence of such factors as steep slopes or poor soils. A distinction is made in the final map between land that is designated open space for recreational uses, such as parks, and land preserved for habitat or species protection. Open space adjacent to an urban area would be rated high if public accessibility was desirable but low if its purpose was resource protection or preservation. Separate maps based on two types of proposed uses present the contrast in analysis.

The analysis of the variables thus was rated as follows:

A. Land with steep slopes and therefore less suited for other purposes.

B. Land that has oak woodland vegetation resources.

C. Riparian land.

D. Low storie index indicating a lack of suitability for agriculture or rangeland.

E. At least approximately one-eighth of a mile from a major road to avoid negative impact on wildlife.

F. Either approximately one-quarter of a mile from urban areas if designated to protect resources or adjacent to urban areas if designated to serve as parks and recreation.

Agriculture

This land use includes all forms of agricultural activity; obviously, its suitability for specific crops and practices would vary. The determination of suitability would need to be made on a site-specific basis.

For the purpose of general agricultural suitability, the highest land suitability for agriculture was a rating of the variables as follows:

A. Land that does not have oak woodland.

B. Land that is not close to perennial creeks (to avoid fertilizer/pesticide runoff contamination).

C. The flattest slopes.

D. The highest storie index.

E. Proximity to a major road (considered an advantage for trucking and farm equipment access).

Rangeland

Some types of livestock can graze under most conditions, but for purposes of this analysis, land more suited for either open space or agricultural designation was rated above that of rangeland. The major limitation to suitability of land for rangeland was a combination of steep slopes and proximity to creeks. A rating of medium for the other variables was considered the most desirable for rangeland purposes.

Details of the Variables

Storie Index

In determining the most suitable uses according to soil fertility, the SCS storie index rating was used, with a modification of the categories to three to accommodate the ratings of high, medium, and low that were used throughout the analysis. Therefore, the first two SCS categories of excellent and good were combined into Category 1. Categories fair and poor were combined into Category 2, and categories very poor, nonagricultural, urban, and mines were combined to compose Category 3.

Subsequently, recoding was undertaken to prioritize these categories according to land use:

Agriculture	High for Category 1
Rangeland	High for Category 2
Open space	High for Category 3

Roads

The five principal arterials of the watershed were used in this analysis:

- Highway 1
- Highway 227
- Los Osos Valley Road
- U.S. 101
- Avila Valley/San Luis

A buffer on each side of the road was created, which was then recoded into the three categories of more than 75 meters, 75 meters to 150 meters, and greater than 150 meters. Each land use was then evaluated according to these criteria, with agriculture deemed the most suitable closest to the roads and open space the least suitable.

Slope

The slope categories in the San Luis Obispo watershed data set were divided into a number of classes, which were recoded into the most appropriate grouping of three classes for the analysis of the three land uses. The existing categories were not altered for the purpose of the demonstration, so they do not necessarily represent the most ideal slopes for the particular uses. A separate category of less than 10 percent slopes was provided for agriculture because most agricultural practices require flat land.

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Slopes between 10 and 21.5 percent would be limited to such activities as orchards or vineyards.

Agriculture:	< 10 percent	High (H)
	10 to 21.5 percent	Medium (M)
	> 21.5 percent	Low (L)
Open space:	< 10 percent	L
	10 to 46 percent	M
	> 46 percent	H

No slope analysis was undertaken for rangeland because this category was combined with that for streams (see Streams section).

Urban Adjacency

Urban adjacency was treated as an overlay of the aggregate map of the other variables because it is an absolute value. That is, this variable has no ranking. Land is either within or outside the buffer. The suitability of each land use adjacent to urban areas was determined, then the aggregate map was adjusted according to a comparison of the recoded aggregate values with the designated ranking of land use suitability.

The first step was to recode the existing data on land uses (interpreted from 1989 aerial photography obtained from the United States Department of Agriculture [USDA], Agricultural Stabilization and Conservation Service, Atascadero, California) into urban/commercial areas and nonurban/noncommercial areas. A buffer zone of approximately one-quarter of a mile was then applied around the urban/commercial areas and an analysis undertaken of suitability for the three land uses to be located within this buffer.

In making the analysis, the following assumptions were made:

Land Use	Ranking	Reason
Agriculture	L	Conflict with dust, noise, pesticides, and urban use
Rangeland	M	Fire hazard of open grassland near buildings
Open space	H	Most suitable if used as parks/recreation
	L	Least suitable if designated for habitat/wildlife protection

Therefore, the analysis provided for two planning alternatives for open space, with the scenarios presented as separate maps overlaid on the aggregate map for the other rangeland variables.

In interpreting this map, the combined values were rated according to the above criteria, with any values lower than the desired ranking receiving a low value no matter what the aggregate value had been. This produced the following results:

- **Agriculture:** A rating of low for any land within the buffer zone.
- **Rangeland**: A rating of medium for any land within the buffer zone except that rated low for the aggregate map.
- **Open Space:** A rating equivalent to the rating of the aggregate map if the land was to be used for parks/recreation, or a rating of low for any land within the buffer zone if the land was to be used for habitat protection.

Streams

The original data for streams (from 1:24,000-scale USGS maps), which included both intermittent and perennial creeks, were used for rangeland analysis. A buffer of 150 meters was then applied to these streams, and these data were combined with the slope analysis. This combination was important in evaluating the erosion hazard and resulting stream pollution caused by nitrogen waste and hoof disturbance. The complete stream complex for the watershed area was therefore evaluated using the following matrix:

Stream Buffer	Slope		
	< 21.5 percent	21.5 to 46 percent	> 46 percent
< 75 meters	1/L	2/M	3/L
75 to 150 meters	4/M	5/H	6/M
> 300 meters	7/H	8/H	9/M

The stream data were then modified for agricultural and open space land use analysis to indicate only the perennial creeks as defined by the California Department of Fish and Game. Buffers were created for this as follows:

Stream Buffer	Open Space	Agriculture
< 150 meters	H	L
150 to 300 meters	M	M
> 300 meters	L	H

Oak Woodlands

The recoding of oak woodland data for agriculture was different than for the other two land uses because the presence of oak woodlands is not conducive to agriculture:

No oak woodlands	H
< 10 percent	M
> 10 percent	L

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The suitability of oak woodlands for open space and rangeland was ranked as follows:

	Open Space	Rangeland
< 33 percent	L	M
33 to 75 percent	M	H
> 75 percent - high	H	L

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R0021895

GIS as a Tool for Predicting Urban Growth Patterns and Risks From Accidental Release of Industrial Toxins

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Introduction

The catastrophic Bhopal incident demonstrated to the world what could happen when industry and population are geographically incompatible. Many believe that the large urban population "should not have been there." A recent publication, "New York Under a Cloud," presents a frightening map of New York State that indicates potential areas of serious population exposure to accidental releases of chemicals stored by area industries and municipalities.

Conventional urban planning and administrative practices at the local level do not adequately provide for the minimization of these risks. Local jurisdictions on the fringes of metropolitan areas may be particularly ill-equipped to respond and plan effectively. Their elected officials, supported by minimal professional staffs and unaware of specific potential risks, may be more interested in soliciting new industrial development along with the tax base it brings. They therefore create industrial zones without restricting facilities that may generate hazardous substances and without recognizing the possibility that underground aquifers, which are current or potential sources of drinking water, may underlie these zones. Jurisdictions often permit facilities that could routinely or accidentally release toxic substances into the air without due regard for prevailing wind patterns or existing or projected urbanized areas that may be affected.

Although the available data and methodology have some gaps, much of the knowledge required to provide adequate protection from these risks exists. We know how to identify the hazardous substances that these sites may produce or store and how to calculate the types and levels of risks associated with them. We can accurately map the locations of streams, underground aquifers, and their catchment areas. Although with less precision, we also can indicate the areas more likely to receive the outfall of airborne and waterborne pollutants.

At the other end of the equation, we can construct models for projecting patterns of urban growth. Those responsible for planning, however, have not made the connections between these techniques. Hazardous facilities sites are thus still permitted in areas that place existing urban residents and their drinking water supplies at risk, and new urban development grows in areas polluted by existing hazardous substance sites.

Clearly, this situation displays a need for the coordinated application of scientific risk assessment techniques and new approaches to regulating urban development. Equally critical, however, is the need to give greater attention to formulating appropriate public policy measures at the local and state levels for dealing with the complex disputes that surround these issues.

Project Background

The project this paper describes addressed these needs. It was undertaken by a team of faculty from the University of Cincinnati's School of Planning, Department of Environmental Health, and the College of Law. The study team focused on the accidental release of hazardous materials both into the air and into the ground-water supply. The team's purpose was to develop an integrated approach to scientific risk assessment, environmental analysis, urban planning, and policy analysis to address conflicts between:

- Expected patterns of suburban residential growth.
- The need to safeguard existing and new residential areas, and their water supplies, from toxic chemical pollution.
- The promotion of industrial development on the peripheries of urban regions, which often leads to the proliferation of hazardous substances sites.
- The need for effective regulation of these sites in complicated multijurisdictional environments.

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This project examined these issues within a 100-square-mile area on the northern edge of metropolitan Cincinnati. The study area is not yet completely urbanized but lies in the path of urbanization. It contains a significant number of industrial or storage facilities that house supplies of hazardous materials. A major aquifer serving as a public water supply source passes under the area. Approximately 17 local jurisdictions fall within the study area: two counties, six townships, and nine municipalities. The area encompasses an intricate mix of agricultural, residential, and commercial land uses. In addition, several major industrial concentrations, as well as a number of jurisdictions, are aggressively soliciting new industrial employment. Because of its proximity to most major employment sites in southwest Ohio and to a variety of large retail complexes, the area is experiencing rapid residential development.

Projecting Areas of Future Development

The study team used PC ARC/INFO geographic information systems (GIS) to project the locations of future residential and industrial growth in the study area, to show the locations of areas at various degrees of risk from either airborne or waterborne industrial toxins, and to reveal the potential areas of population exposure resulting from the overlap of these areas.

Although this paper does not describe the models used to project residential and industrial growth in the study area, it does include the criteria used to make projections. The criteria we used to project residential growth were:

- Travel times to major employment concentrations in the region.
- Proximity to interstate highways, interchanges, and main trunk sewers.
- Avoidance of areas composed of steep slopes and flood plains.
- Land currently zoned for agriculture or housing.

The criteria for projecting industrial areas were:

- Relatively flat, not in a flood plain, and zoned industrial.
- Proximity to existing industrial development, main trunk sewers, and interstate highways.
- Relative aggressiveness of local jurisdictions in attracting industrial development.

Identifying Areas at Risk From Airborne Releases

The study team determined the model project areas at risk from airborne releases by using information available from the Ohio Environmental Protection Agency. This information, which included the location, identity, and quantity of hazardous materials recently stored in the study area, was available as a result of reporting requirements mandated by several federal statutes. The study team assumed that a similar ratio reflecting sites containing hazardous materials to the area of industrially developed land would continue into the future. Based on this assumption, the study team could randomly project new potential release sources.

The algorithm and associated software used for calculating the plume size of aerial dispersion of hazardous chemicals was Aerial Locations of Hazardous Atmospheres (ALOHA). The National Oceanic and Atmospheric Administration (NOAA) developed this system, which is in wide use by government and industry for the preparation of emergency contingency plans. This software is available for use on any Macintosh computer, or any IBM-compatible with an Intel 80286 (or better) CPU. This software employs three classes of variables in calculating the plume dispersion for a specific chemical:

- Chemical variables
- Meteorological options
- Source strength options

Chemical variables include both physical properties of the chemical and parameters that define the human health effects of the chemical. In the latter case, the two variables are the threshold limit value (TLV) and the immediately dangerous to life and health (IDLH) value. The TLV is a measure of chronic toxicity of the chemical in humans. It represents the maximum concentration of the chemical in air to which a human can safely be exposed for 8 hours per day on a daily basis. The IDLH is a measure of acute toxicity of the chemical in humans. The IDLH represents the maximum concentration to which a human can be exposed for a short time and not experience death or some other severe endpoint.

Meteorological options describe the ambient atmospheric conditions into which the chemical disperses. ALOHA has the capability of downloading real-time data from NOAA satellites. This case, however, employed average meteorological conditions for the study area over the course of a year. The variables this study used were atmospheric inversion height (or no inversion), wind velocity, air temperature, ground roughness (rural or urban), and stability class (a combined variable describing cloud cover and incoming solar radiation).

The source options quantify the amount of chemical being released and describe how the chemical is released (instantaneous or continuous).

The ALOHA model provides a procedure for showing the IDLH and TLV risk zones from a single accidental release of a single chemical, given specified conditions of atmospheric stability, wind direction, and air temperature. Obviously, climatic conditions change daily, so areas surrounding a single industrial site experience

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different degrees of risk depending on the variability of these conditions. Moreover, a single site that can potentially release more than one chemical poses a higher risk to surrounding areas. Finally, when release plumes from two or more sites that are located relatively close to each other overlap, risks also increase. To account for these factors of climatic variation and overlapping chemical release plumes, the study team constructed the model described below.

In discussing these procedures, bear in mind that the risk factors are relative. Sufficient data are not available to estimate the absolute probability of an accidental release of industrial toxins into the air. Consequently, no absolute risk levels can be estimated.

1. We acquired NOAA climatic statistics for a full year for the weather station nearest the study. NOAA tabulates eight wind directions (N, NE, E, SE, S, SW, W, and NW). We sorted the average daily wind speeds and average daily temperatures according to the daily prevailing wind direction. Thus, for each of the eight wind directions, it was possible to determine the number of days in the year that the prevailing wind comes from that direction, as well as the average daily wind speed and temperature.

2. We then input the temperature and wind speed data, derived as explained above, into the ALOHA model to prepare plots of the IDLH and TLV zones, or plumes, for each of the eight wind directions. Individually, the plumes emanated downwind from the source of the release. In our study, IDLH plumes varied from 0.17 miles to greater than 10 miles. TLV plumes varied from 0.52 miles to greater than 10 miles.

   When the plumes for the eight different wind directions were combined, the results were translated into a pattern of wedges representing various plume lengths in each of the eight different wind directions. These risk levels vary according to the number of days per year the wind blows in each direction from the source of the release. Plumes blowing in different directions vary in length according to average temperature and wind velocities for the days the wind blew in each direction. The numbers in the wedges represent risk factors assigned as indicated in Table 1.

Table 1. Assigned Risk Factors

Frequency of Wind	Risk Factor
0-25 days/year	1
26-50 days/year	2
51-75 days/year	3
76-100 days/year	4

3. When a single industrial location employed more than one chemical, the IDLH and TLV risk patterns for each were overlaid on one other. Where the overlap occurred, we added the risk factors together.

4. Finally, when the risk patterns from two or more sites overlapped, we added together the risk factors assigned to overlapping areas. The overlap capabilities of GIS allowed us to easily draw and combine the risk patterns, superimposed on a map of the industrial sites under consideration.

When we compared the eight existing combined IDLH and TLV risk patterns with the risk patterns that appear when six projected new sources of releases and projected areas of residential growth are added, the changes are rather dramatic. Of course, we must note that the projected sources of potential releases are hypothetical and their locations selected at random. In reality, precise locations of areas at greater risk cannot be predicted with any degree of certainty. We can reasonably deduce from the maps, however, that any substantial increase in industries storing or using toxic chemicals that might be accidentally released into the air can compound risk levels much more than might be expected. The maps we produced indicated five risk levels.

The study results also showed that an industry capable of generating accidental releases of airborne toxins will very likely place at risk residents not only of the same community but those in neighboring jurisdictions as well. This is particularly true on the fringes of metropolitan areas where highly fragmented political boundaries exist. This fact complicates tremendously the ability of each jurisdiction to protect its citizens and suggests a need for a more comprehensive approach to regulation than conventional land use zoning measures that each locality administers.

Identifying Areas at Risk From Accidental Releases Into Ground-Water Supplies

As the introduction of this paper indicates, local government usually manages conventional land use planning, while air quality is largely a state or federal responsibility. Thus, decisions regarding the regulation of industrial location may not account for the types of industrial operations proposed, the possible use of hazardous materials, and possible risks to local residents from accidental release of toxins into the air. The same problem applies to the location of industries that have the potential for accidental release of hazardous materials into ground-water supplies. In our study, we outlined a technique for predicting where local residents may be placed at risk by drinking water from sources vulnerable to contamination by industrial toxins.

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As stated earlier, the study area includes a major aquifer. We noted industrial and related facilities that use and store hazardous materials, and that are located over or immediately adjacent to the area aquifers. We distinguished between sites where a previous spill had been reported, those where a waste well is located, and all other locations where a hazardous material is used or stored. The results clearly indicated a significant potential for contamination.

To predict the number and locations of additional industrial toxin sources that might appear over a 10-year period following 1990 (the base year of the study), we employed a procedure similar to that used in the air pollution section of the study. We assumed that during this period, the number of such sites would increase by 35 percent. The 35-percent increase represented an arbitrarily selected figure approximately midway between estimates of 25- and 50-percent industry growth in the study area. We used this increment to project additional waste well sites and sites that would experience a spill sometime during the decade, as well as the total number of new sites.

We projected the locations of the additional sites by overlaying a 5,000-foot by 5,000-foot grid on the study area and assigning the new sites to grid cells using a random number generator. We considered only cells lying completely or largely over the aquifers and also falling within the area of projected industrial land use. Each cell was assigned a relative contamination risk factor based on the number of projected sites it contained, with a multiplier of 3 applied to sites with a waste well and a multiplier of 4 applied to sites assumed to have had spills. We also assigned existing sites to grid cells and scored them in the same manner.

To obtain more information, we used a simplified version of DRASTIC maps prepared by the Ohio Department of Natural Resources. DRASTIC is an acronym for **D**epth to water, net **R**echarge, **A**quifer media, **S**oil media, **T**opography, **I**mpact of the vadose zone, and hydraulic **C**onductivity of the aquifer. These factors contribute to an index of the relative vulnerability of the aquifer to pollution. Different shades on the map represented the relative vulnerability of sections of the aquifers to ground-water pollution. We assigned two points to aquifers with a DRASTIC pollution potential index less than 180 and four points to aquifers with a higher index. This allowed us to use the GIS to combine the DRASTIC vulnerability map with the maps that showed risks of pollution from the hazardous materials sites. The resultant maps showed the existing and projected potential risk of pollution in different areas of the aquifers.

The next step in the study related the above information to public water companies that extract water from different parts of the aquifers. Thus, we were able to associate risk levels with well locations as well as with the areas served by water companies whose supplies may be at risk. In this case, the projected risks of polluted water supplies were identical to existing risk levels because no public water wells happened to be located in areas where additional potential sources of pollution were projected.

Of course, wide areas in the study area had no public water supply. Residents in these areas may be at risk depending on where they dig private wells. The maps we generated showing areas at risk nonetheless provide useful guides to potentially hazardous areas.

As in the construction of all such models, we needed to make a number of assumptions, simplifications, and value judgments. In this case, these included the projected number of new toxic sites and point scores assigned to hazardous materials sites and the various areas in the DRASTIC maps. Also, for the sake of simplicity, we projected no new well sites in preparing one of the maps. This additional element should probably be included, assuming local water companies could provide projected well locations. Use of a GIS, however, makes it possible to explore the implications of adjustment of any of these factors.

We must note two more significant omissions from the model that the GIS cannot factor in. One was our inability to identify from the data specific chemicals that each sight might release and the relative effects of each. We would have required more complicated techniques for dealing with these variables and for projecting the travel and dilution of plumes of contaminants in an aquifer. In the interest of providing a simple, if relatively crude, model capable of replication by a local planning agency with a simple GIS, we elected not to propose use of more sophisticated techniques.

Another obvious omission was the consideration of water pumping and treatment measures that might mitigate risks of water contaminated by accidental release of industrial toxins. Perhaps, with knowledge of the specific contaminants found in the water at any given time, such mitigation might be possible. We elected not to consider this factor for reasons of simplification but also because this study aimed to provide planning agencies the means to identify potential risks to local residents and to prevent or minimize them through better land use planning and regulatory measures.

Some Final Notes

As stated earlier, the purpose of the study was to propose a technique that planning agencies could use to identify:

- The locations of existing and projected patterns of residential and industrial development in a multijurisdictional suburban area.

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- The locations of existing and projected industrial, storage, and disposal sites of hazardous materials.
- Residential areas that might be placed at risk by the accidental release of these hazardous materials into the air or into ground-water supplies.
- The relative levels of risks resulting from potential exposure to more than one hazardous material at a single site, or from multiple sites in the vicinity.

The technique we used in the study permitted projection of *relative* risk levels. Projecting absolute risk levels is impossible without data on the actual incidence of accidental releases of toxins over time in this or in similar areas. A related question is whether it is possible to meaningfully indicate the combined risk from airborne releases of industrial toxins and drinking water contamination to a particular residential area. The issue of weighting relative risk factors is central here. Could we, for example, weight the risk levels of exposure to airborne releases three—or possibly four—times higher than water contamination risks? Or can we say that the risks may be the same, but the danger from airborne releases is three or four times greater?

Obviously, these would be futile exercises, especially because the point scores in the separate mapping studies were arbitrarily assigned. The public should know, however, which present or projected residential areas carry some level of risk from both types of exposure. Thus, after calculating point scores to derive relative risk levels from waterborne pollutants, we multiplied the scores by 4 to bring their maximum ranges into the same order of magnitude. Otherwise, the effect of waterborne pollutants would not be apparent. Consequently, we created two maps to show the combined existing and projected risks; therefore, we highlighted the combined risks that residents face from both airborne and waterborne hazardous materials.

The maps of airborne releases used in these combinations showed TLV. Continuing exposure within the TLV areas over an extended period can also have adverse health effects. This study focused only on accidental releases, however, and a single release is unlikely to sustain continuous exposure. Of course, residential areas at risk from several sites might approach conditions of sustained exposure. This situation is more analogous to prolonged exposure to contaminated drinking water supplies. We did not combine the maps of IDLH airborne releases with the maps of areas at risk from ground-water contamination because they are not analogous conditions. Nonetheless, the IDLH risk maps in themselves reveal the conditions that local planning officials should most seriously consider.

Replicating the procedures outlined in this study should be technically and financially feasible for local planning agencies. Armed with the results of such an investigation, their next step should be to establish the planning and regulatory measures that would minimize both existing and projected levels of risk to area residents. The attorney on our team has outlined a range of possible measures, but detailing them would be the subject of another paper.

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Integration of GIS and Hydrologic Models for Nutrient Management Planning

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Introduction

Recent evidence that agriculture in general, and animal waste in particular, may be an important factor in surface and ground-water quality degradation has increased the interest in nutrient management research. The presence of nitrogen and phosphorus in surface water bodies and ground water is a significant water quality problem in many parts of the world. Some forms of nitrogen and phosphorus, such as nitrate N and soluble P, are readily available to plants. If these forms are released into surface waters, eutrophic conditions that severely impair water quality may result. Advanced eutrophication (pH variations, oxygen fluctuations or lack of it in lower zones, organic substance accumulation) can cause physical and chemical changes that may interfere with recreational use and aesthetic appreciation of water. In addition, possible taste and odor problems caused by algae can make water less suitable or desirable for water supply and human consumption (1).

Increases in nutrient loadings to water resources have recently been observed in the southeastern United States, where well-drained sandy soils with low nutrient retention capacity and high water table conditions are found in most coastal areas. Those increases were associated statistically with nutrient sources such as agricultural fertilizers and dense animal populations (2, 3). Repetitive occurrences of extensive blooms of blue-green algae that threatened the overall health of Lake Okeechobee, located in southern Florida, were attributed to an increase in nutrient loadings to the lake. The South Florida Water Management District (SFWMD) reported an increase of phosphorus concentrations in the lake water from an annual average of 0.049 milligrams per liter in 1973 and 1974 to a peak of 0.122 milligrams per liter in 1988 (4).

Most water quality problems concerning phosphorus result from transport with sediment in surface runoff into receiving waters. Continuous high loadings from animal waste on sandy soils with low retention capacity, however, may contribute significant quantities of labile phosphorus to subsurface drainage. Ground-water aquifers may also become polluted due to recharge of high loadings of nitrogen. Drinking water with nitrate N-concentrations higher than 10 milligrams per liter may lead to methemoglobinemia in infants. Ground-water monitoring of the Middle Suwannee River area in Florida has shown high concentrations of nitrate nitrogen near intensive agricultural operations. The U.S. Geological Survey has intensively monitored dairy and poultry farms and has found high nitrate levels below these operations compared with nearby control wells (5).

Animal waste management has always been a part of farming, but historically has been relatively easy due to the buffering capacity of the land. In fact, land application of animal waste at acceptable rates can provide crops with an adequate level of nutrients, help reduce soil erosion, and improve water holding capacity. As the animal industry attempts to meet the food requirements of a growing population, however, it applies new technologies that reduce the number of producers, but create larger, more concentrated operations. That, in addition to the decreasing amount of land available for waste application, has increased the potential for water quality degradation.

Successful planning of an animal waste management system requires the ability to simulate the impact of waste production, storage, treatment, and use on water resources. It must address the overall nutrient management for the operation, including other nutrient sources such as supplemental fertilizer applications. Livestock operations are highly variable in their physical facilities, management systems, and the soil, drainage, and climatic conditions that affect the risk of water pollution from animal wastes (6). Linkage between geographic information systems (GIS) and hydrologic models offers an excellent way to represent spatial features of the fields being simulated and to improve results. In addition, a GIS containing a relational database is an excellent way

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to store, retrieve, and format the spatial and tabular data required to run a simulation model.

This paper examines some of the issues related to the integration of hydrologic/water quality models and GIS programs. In addition, the paper discusses the approaches used in the Lake Okeechobee Agricultural Decision Support System (LOADSS), which was recently developed to evaluate the effectiveness of different phosphorus control practices (PCPs) in the Lake Okeechobee basin. The paper also details a dairy model, designed to simulate and evaluate the impacts of alternative waste management policies for dairy operations, that is currently under development.

Hydrologic Models and GIS

By using models, we can better understand or explain natural phenomena and, under some conditions, make predictions in a deterministic or probabilistic sense (7).

A hydrologic model is a mathematical representation of the transport of water and its constituents on some part of the land surface or subsurface environment. Hydrologic models can be used as planning tools for determining management practices that minimize nutrient loadings from an agricultural activity to water resources. The results obtained depend on an accurate representation of the environment through which water flows and of the spatial distribution of rainfall characteristics. These models have successfully dealt with time, but they are often spatially aggregated or lumped-parameter models.

Recently, hydrologists have turned their attention to GIS for assistance in studying the movement of water and its constituents in the hydrologic cycle. GIS programs are computer-based tools to capture, manipulate, process, and display spatial or georeferenced data. They contain geometry data (coordinates and topological information) and attribute data (i.e., information describing the properties of geometrical objects such as points, lines, and areas) (8). A GIS can represent the spatial variation of a given field property by using a cell grid structure in which the area is partitioned into regular grid cells (raster GIS) or by using a set of points, lines, and polygons (vector GIS).

A close connection obviously exists between GIS and hydrologic models, and integrating them produces tremendous benefits. Parameter determination is currently one of the most active hydrology-related areas in GIS. Parameters such as land surface slope, channel length, land use, and soil properties of a watershed are being extracted from both raster and vector GIS programs, with a focus on raster-based systems. The spatial nature of GIS also provides an ideal structure for modeling. A GIS can be a substantial time saver that allows different modeling approaches to be tried, sparing manual encoding of parameters. Further, it can provide a tool for examining the spatial information from various user-defined perspectives (9). It enables the user to selectively analyze the data pertinent to the situation and try alternative approaches to analysis. GIS has been particularly successful in addressing environmental problems.

Approaches for Integrating GIS and Models

A significant amount of work has been done to integrate raster and vector GIS with hydrologic/water quality models. Several strategies and approaches for the integration have been tried. Initial work tended to use simpler models such as DRASTIC (10) and the Agricultural Pollution Potential Index (11). In these cases, the models were implemented within the GIS themselves. These studies attempted to develop GIS-based screening methods to rank nonpoint pollution potential. The use of more complex models requires that the GIS be used to retrieve, and possibly format, the model data. The model itself is implemented separately and communicates with GIS via data files. Goodchild (12) refers to this mode as "loose coupling," implying that the GIS and modeling software are coupled sufficiently to allow the transfer of data and perhaps also of results, in the reverse direction. Fedra (8) refers to this level of integration as "shallow coupling" (see Figure 1). Only the file formats and the corresponding input and output routines, usually of the model, must be adapted. Liao and Tim (13) describe an application of this type, in which an interface was developed to generate topographic data automatically and simplify the data input process for the Agricultural Nonpoint Source (AGNPS) Pollution Model (14), a water quality model.

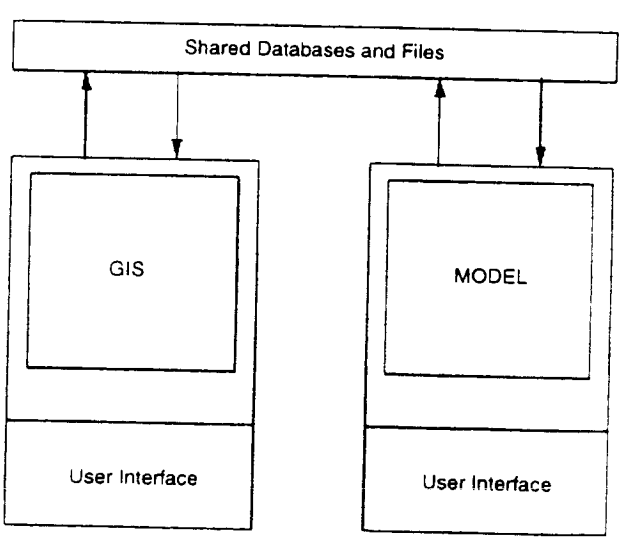

Figure 1. Loose or shallow coupling through common files (8).

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Higher forms of connection use a common interface and transparent file or information sharing and transfer between the respective components (see Figure 2). The dairy model, currently under development, is an application of this kind. It will link the Ground-Water Loading Effects of Agricultural Management Systems (GLEAMS) (15) model and GIS to evaluate potential leaching and runoff of both nitrogen and phosphorus.

LOADSS is an extension of this type of application because it includes an optimization module that enables the system to select the best PCPs at the regional scale, based on goals and constraints defined by the user.

Both applications use ARC/INFO's arc macro language (AML), a high-level application language built into the GIS. A subset of functions of a full-featured GIS, such as creation of maps (including model output) and tabular reports, as well as model-related analysis, are embedded in the applications, giving the system great flexibility and performance. Fedra (8) describes a deeper level of integration that would merge the two previous approaches, such that the model becomes one of the analytical functions of a GIS, or the GIS becomes yet another option to generate and manipulate parameters, input and state variables, and model output, and to provide additional display options. In this case, software components would share memory rather than files.

The choice between integrating a water quality model with a raster or vector GIS depends on the importance of spatial interactions in the process being studied and the nature of the model itself. Some water quality models, such as GLEAMS, are field-scale models that provide edge-of-the-field values for surface runoff and erosion as well as deep percolation of water and its constituents. In this case, spatial interactions between adjacent fields are ignored and a vector GIS can be used to describe the system. Moreover, important factors in the simulation process, such as land use and management practices, are normally field attributes and thus, are better represented in a vector structure.

Other factors playing an important role in the hydrologic process, such as field slope, aspect, and specific catchment area, are hard to estimate in vector systems, however. A raster-based GIS is better suited for handling watershed models in which the routing process is important and spatial interactions are considered. For those, several algorithms for estimating important terrain attributes are often incorporated in commercially available raster-based GIS programs.

LOADSS

LOADSS was developed to help address problems created by phosphorus runoff into Lake Okeechobee. It was designed to allow regional planners to alter land uses and management practices in the Lake Okeechobee basin, then view the environmental and economic effects resulting from the changes. The Lake Okeechobee basin coverage incorporates information about land uses, soil associations, weather regions, management practices, hydrologic features, and political boundaries for approximately 1.5 million acres of land and consists of close to 8,000 polygons.

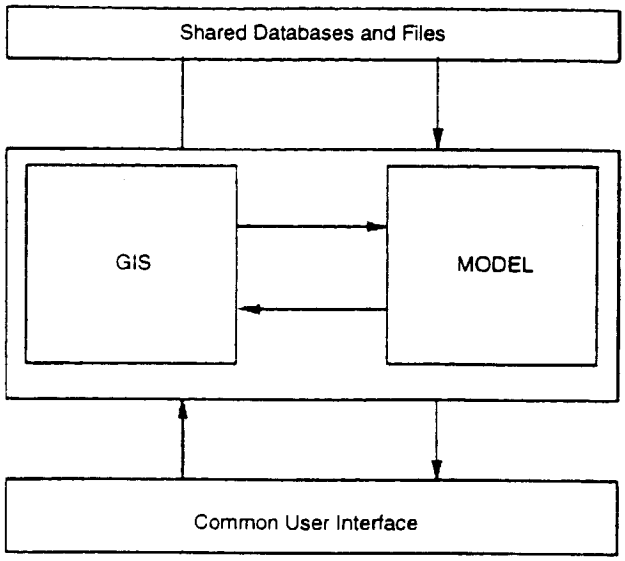

Figure 2. Deep coupling in a common framework (18).

The SFWMD, responsible for managing Lake Okeechobee, has initiated numerous projects to develop effective control practices for reducing the level of phosphorus in agricultural runoff as part of the Lake Okeechobee Surface Water Improvement and Management (SWIM) Plan. These projects, numbering more than 30, were designed to develop information on the control and management of phosphorus within the lake basin and to determine the costs and effectiveness of selected management options. Three types of control options are being studied:

- Nonpoint source controls, such as pasture management.
- Point source controls, such as sewage treatment.
- Basin-scale controls, such as aquifer storage and retrieval.

After completing most of these research efforts, the need arose for a comprehensive management tool that could integrate the results for all three classes of PCPs. In response to these needs, design and implementation of a decision support system was initiated with the following objectives (16):

- Organize spatial and nonspatial knowledge about soils, weather, land use, hydrography of the lake basin, and PCPs under a GIS environment.

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- Develop and implement algorithms for modeling nonpoint source, point source, and basin-scale PCPs.
- Develop and implement mechanisms for evaluating the performance of the entire Lake Okeechobee basin under different combinations of PCPs applied to the basin.
- Design and develop a user interface that would facilitate use of the system by noncomputer experts.

The goal in developing LOADSS was to create an information system that would integrate available information to help regional planners make decisions. LOADSS can generate reports and maps concerning regional land attributes, call external hydrologic simulation models, and display actual water quality and quantity sampling station data. LOADSS is a collection of different components:

- The regional scale GIS-based model used to develop and manipulate regional plans for reducing phosphorus loading to Lake Okeechobee.
- The Interactive Dairy Model (IDM) used to develop field-level management plans for dairies and run the Field Hydrologic and Nutrient Transport Model (FHANTM) simulation model for nutrient transport modeling.
- An optimization module that enables the system to select the best PCPs at the regional scale (currently under development).

Although these components can run independently, they are fully integrated in the LOADSS package and can exchange information where necessary. A design schematic of LOADSS is given in Figure 3.

Regional-Scale GIS-Based Model

LOADSS serves both as a decision support system for regional planning and as a graphic user interface for controlling the different components. One consideration in the design of LOADSS was the size of the database that was being manipulated. Because the land use database consisted of nearly 8,000 polygons, running the simulation models interactively would not be a feasible option. Thus, the CREAMS-WT (17) runoff model was prerun for different levels of inputs and management for each land use, soil association, and weather region (18).

Depending on the land use and its relative importance as a contributor of phosphorus to the lake, anywhere from one (background levels of inputs to land uses like barren land) to 25 (dairies, beef pastures) levels of inputs were selected. Each set of inputs to a particular land use was given a separate PCP identification code. A CREAMS-WT simulation was performed for each PCP, on each soil association and weather region. This resulted in approximately 2,600 simulation runs. Annual average results were computed for use in LOADSS. CREAMS-WT provides an average annual estimate of phosphorus runoff from each polygon. Phosphorus assimilation along flow paths to Lake Okeechobee are estimated as an exponential decay function of distance traveled through canals and wetlands (4).

The imports, exports, and economics of each PCP are based on a per production unit basis. Depending on the type of polygon, the production unit can be acres (e.g., pastures, forests), number of cows (dairies), or millions of gallons of effluent (waste treatment plants and sugar mills). Developing a regional plan in LOADSS involves assigning a PCP identification code to each one of the polygons in the Lake Okeechobee basin. Accessing the results of a regional plan involves multiplying the production unit of each polygon by its appropriate database import, export, or economic attribute and summing the resulting values over all polygons in the Lake Okeechobee basin. LOADSS runs in the ARC/INFO Version 6.1.1 GIS software on SUN SPARC stations.

Interactive Dairy Model

Although the LOADSS level of detail is adequate for regional planning, a more detailed model was necessary to analyze individual dairies in the Lake Okeechobee basin, as dairies were one of the large, concentrated sources of phosphorus runoff into the lake. Thus, the IDM was developed and incorporated into LOADSS. IDM utilizes FHANTM to simulate phosphorus movement in dairy fields. FHANTM is a modification of DRAINMOD (19) with added functions to handle overland flow routing, dynamic seepage boundary, and soluble phosphorus algorithms for P input, mass balance, and transport (20).

Unlike in LOADSS, FHANTM is run interactively, as IDM requires. Furthermore, in LOADSS, the user can only select from a number of predefined PCPs, while in IDM, the user has access to more than 100 input and management variables, all of which can take a range of values. This allows for the development and evaluation of detailed dairy management plans that otherwise would be impossible at a regional scale. While LOADSS only provides average annual results, IDM displays daily time series simulation results. IDM uses the same assimilation algorithm and can produce the same phosphorus budget maps and reports as LOADSS.

Optimization Module

A variety of factors must be considered in planning nutrient management programs. Production and environmental goals need to be balanced, and these goals are often incompatible. Performing this exercise on a regional scale, comprising many fields for which a variety of land uses and management options can be assigned, is a tremendously time-consuming, if not impossible,

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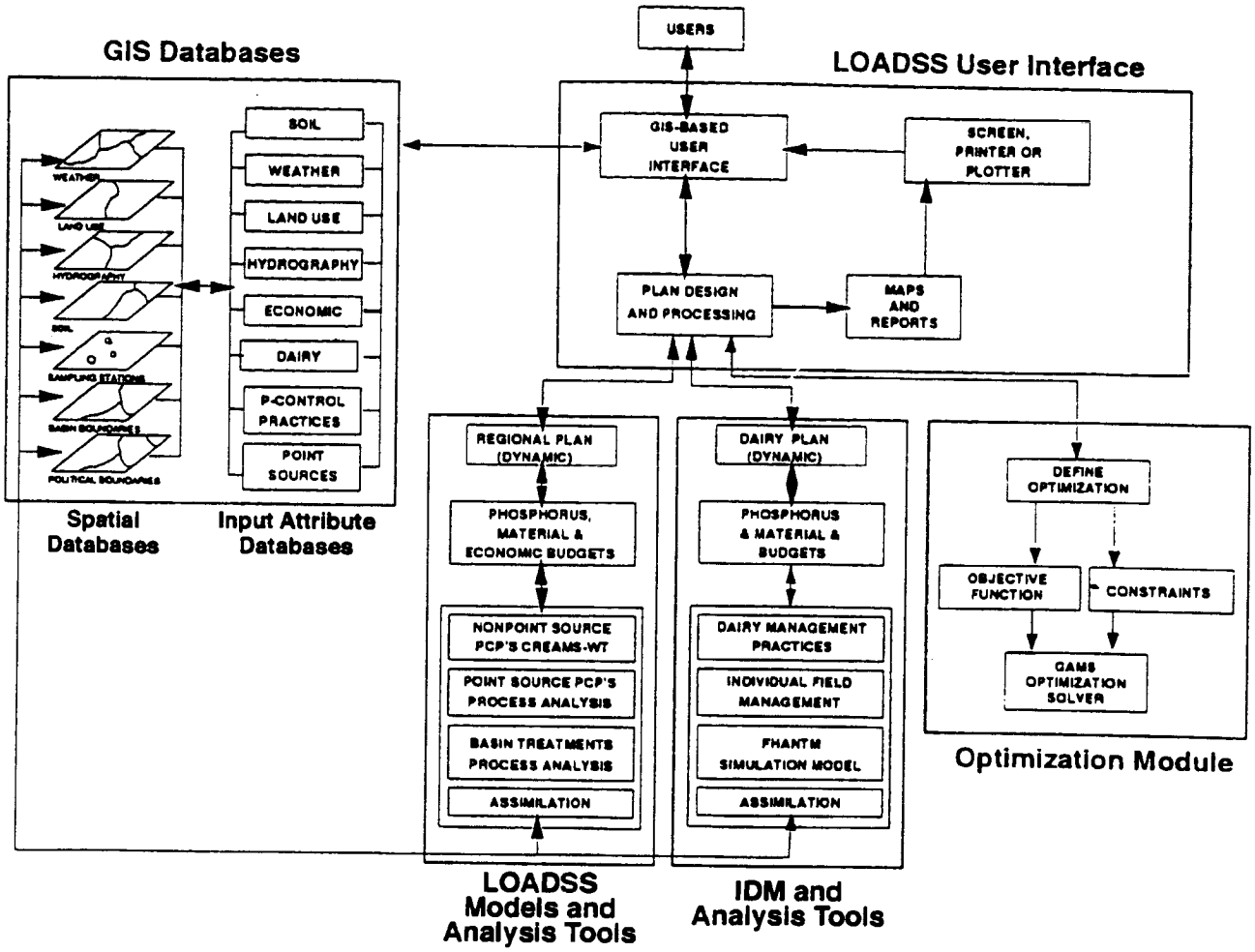

Figure 3. LOADSS design schematic (16).

task. The optimization component of LOADSS, currently under development, will determine the best combination of agricultural, environmental, and regulatory practices that protects and maintains the health of Lake Okeechobee and also maintains the economic viability of the region. The optimization process will provide another method for modifying the PCPs assigned to individual fields. Different optimization solution methods, such as linear programming and integer linear programming, will be available for solving the optimization problem that the user defines.

Dairy Simulation Model

The dairy model was expected to be fully functional by the end of 1994. It is designed to be an additional tool for answering questions related to the environmental costs and impacts of dairy operations. A design schematic of the dairy model is given in Figure 4. It differs from the LOADSS/IDM model in the following aspects:

- It is designed to be generic so that any dairy represented by a coverage for which relevant data, such as topography, soil characteristics, weather, and field boundaries, are available can be simulated.
- The GLEAMS water quality model will be used for simulating nutrient transport of nitrogen and phosphorus.
- The user will be able to assign a larger variety of crops and crop management practices to the individual fields, including crop rotation.

GLEAMS (15) is a field-scale water quality model that includes hydrology, erosion/sediment yield, pesticide, and nutrient transport submodels. GLEAMS was developed to use the management oriented CREAMS (21) model and incorporate more descriptive pesticide subroutines and more extensive treatment of the flow of water and chemicals in the root zone layers. The water is routed through computational soil layers to simulate the percolation through the root zone, but the volume of percolation in each layer is saved for later routing in the pesticide component. A minimum of three and a maximum of 12 layers with variable thickness may be used. Soil parameter values are provided by soil horizon, and

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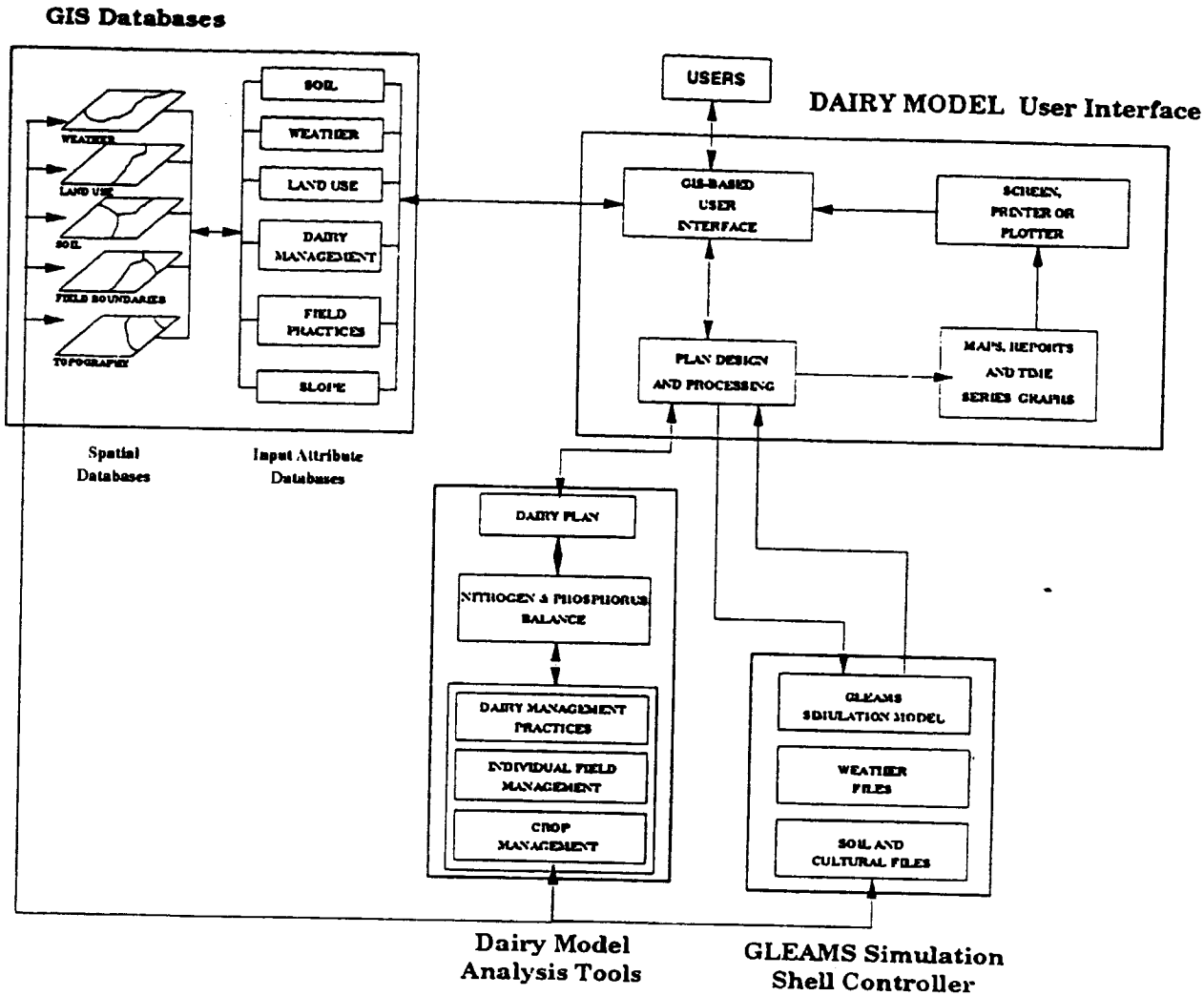

Figure 4. Dairy model design schematic.

the crop root zone may have up to five horizons. The values for parameters, such as porosity, water retention properties, and organic content, are automatically fitted into the proper computational layers. Two options are provided in the model to estimate potential evapotranspiration, the Priestly-Taylor method (22) and the Penman-Monteith method (23). The nutrient component of the model simulates land application of animal wastes by creating appropriate nitrogen and phosphorus pools for mineralization. It considers ammonia volatilization from surface-applied animal waste by using a relationship developed by Reddy et al. (24).

The graphic interface is designed to help the user plan a balanced nutrient management program for the dairy being simulated. First, total nutrient production and accounting are estimated, based on information related to the dairy management such as herd size, confinement system, waste characterization, and handling. Figure 5 shows the general structure of the graphic interface and a first version of the menu used to estimate the total amount of nitrogen and phosphorus available for assignment to the various fields. Nutrient losses during waste storage and treatment vary widely depending on the method of collection, storage, and treatment. Climate can also have a great effect on the losses. Covering all possible methods of storage and treatment is practically impossible, especially in an application that is designed to be generic and applied in any part of the country. A simplification was made to overcome this problem: the user must provide the percentage of original nitrogen and phosphorus that is retained after waste storage and treatment. The menus designed to enter information related to the management of fields and crops are given in Figure 6.

For each field, a sequence of crops can be defined in the Field Management Table, and for each crop, the sequence of practices or field operations is defined in the Crop Management Table. Every time a waste application operation is defined or a field is used as pasture for a certain period, the corresponding amount of nutri-

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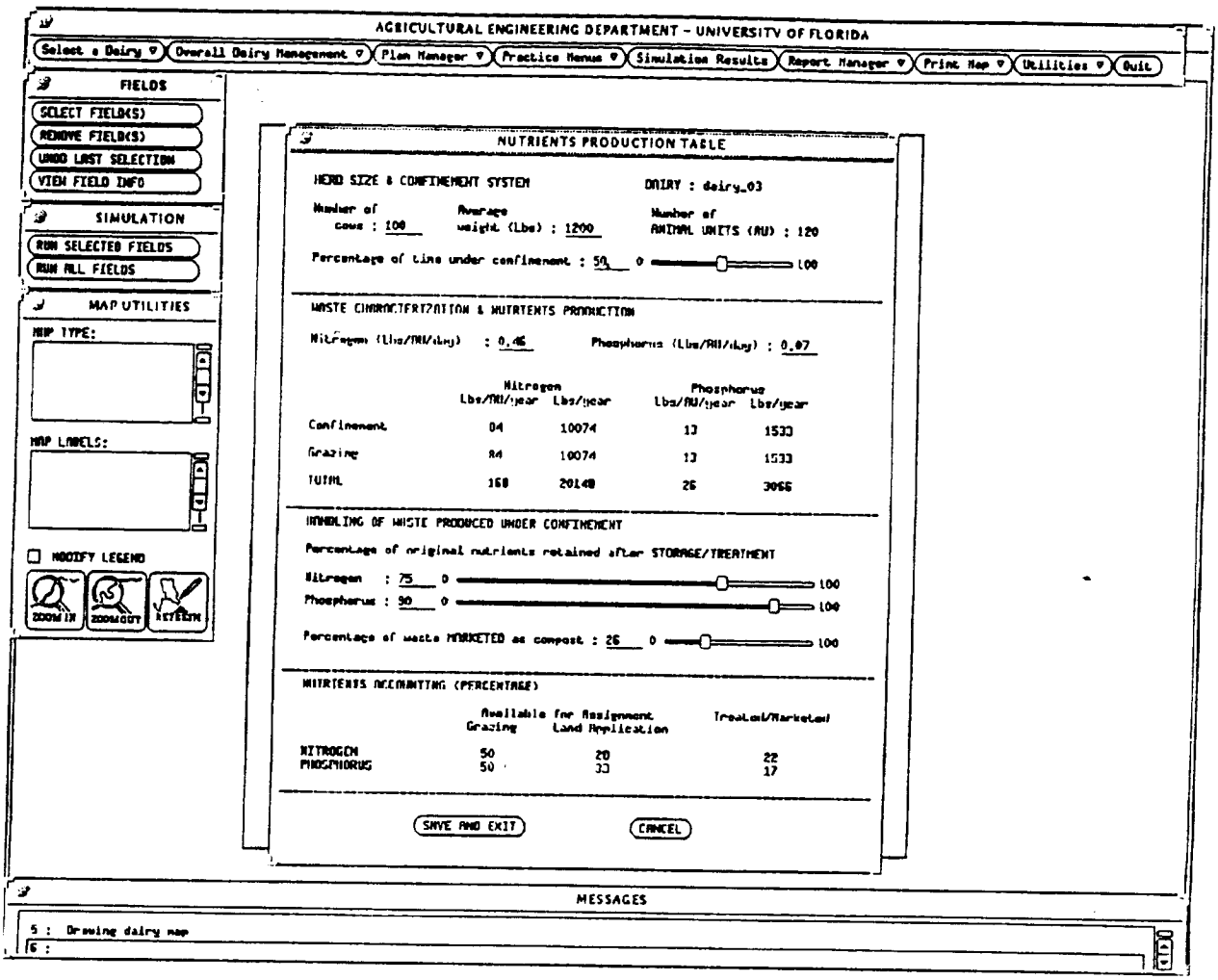

Figure 5. The nutrient production table of the dairy model interface.

ents will decrease from the amount available for assignment and the total available for future assignment will be updated. Once the total amount of nutrients is assigned, the model can be run for the several fields in the dairy and the results evaluated in terms of nutrient loadings to the edge of fields and ground water. Alternative plans can be designed and saved for comparison and selection of best management options. The best solutions in terms of reducing nutrient loadings to surface and ground water must also consider economic aspects. A producer's decision about competing waste management practices is ultimately economically motivated. Thus, the system will eventually include a tool for economic analysis of alternative management options.

Summary and Conclusions

The search for solutions to the many problems concerning nutrient management that affect water resources implies a continued demand for the development of modeling systems that can be used to analyze, in a holistic approach, the impact of alternative management policies.

The development of LOADSS exemplifies how the integration of hydrologic models and a GIS can be used for analyzing nutrient control practices at different scales. The addition of optimization algorithms further enhances the ability of policy- and decision-makers to analyze the impact of alternative management practices and land uses at the regional level.

The first part of LOADSS (Version 2.2) that includes the CREAMS-WT regional-scale model and the IDM components is fully functional and currently available at the SFWMD. Preliminary results show that LOADSS behaves consistently with measured data at the lake basin scale. Some of this, however, is due to offsetting errors in model behavior at the subbasin scale, particularly in subbasins that are adjacent to or very far from the lake. Currently, projects are underway to further verify and calibrate the model at the subbasin level to improve its

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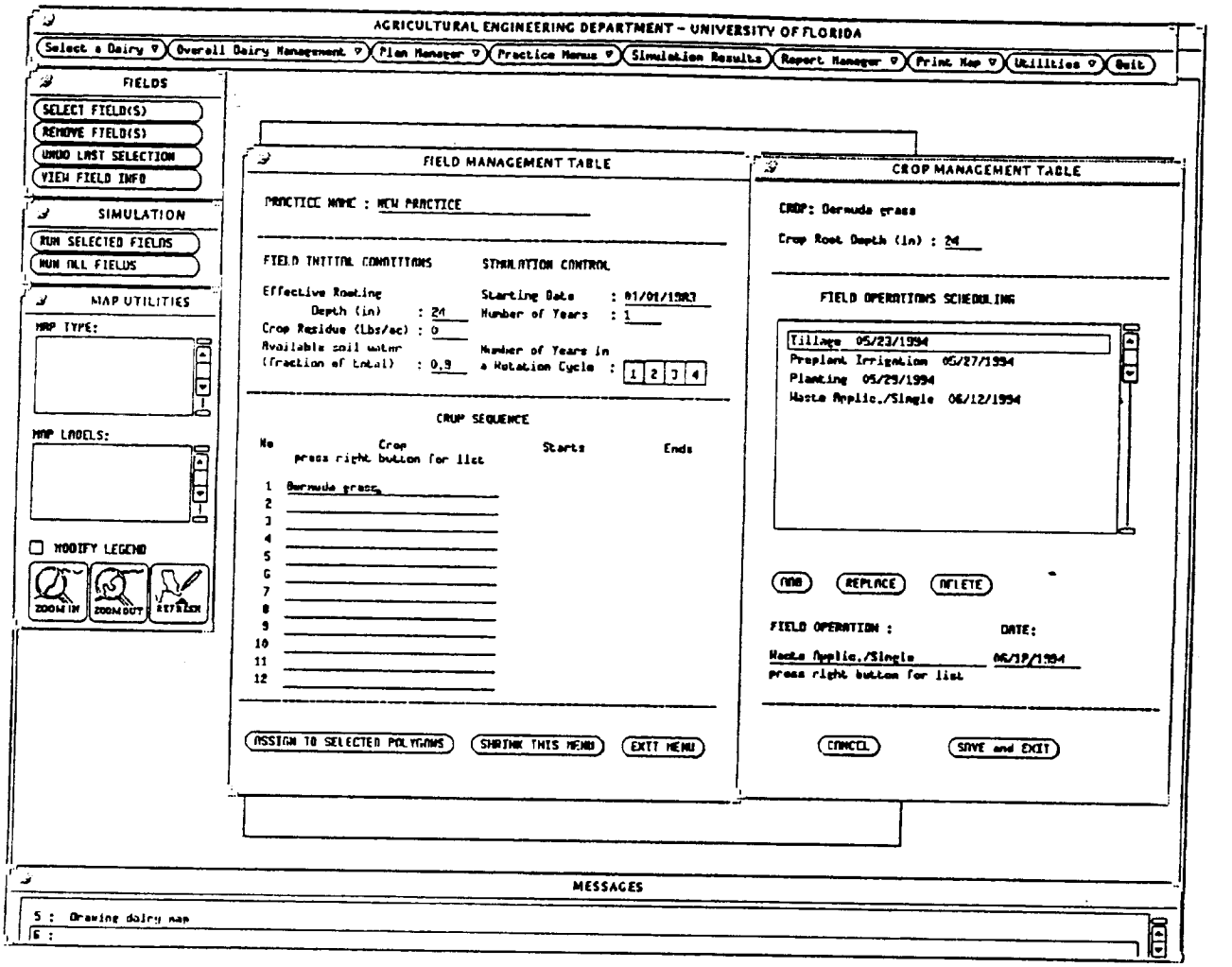

Figure 6. Field and crop management tables of the dairy model interface.

performance at smaller scales (16). Initial results of the optimization component are currently being evaluated.

The dairy model represents a different approach in integrating water quality models and GIS in that it is designed to be generic and focused mainly on the farm level. It is primarily designed to help policy- and decision-makers analyze the effects of alternative dairy waste management practices on the farm level. The framework can easily be adapted to handle different types of animal wastes (such as beef cattle and poultry) and to simulate the impact of other crop management practices such as pesticide applications.

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Other GIS Applications

Expedition of Water-Surface-Profile Computations Using GIS

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Abstract

Water-surface profiles computed by use of a step-backwater model such as Water Surface PROfile (WSPRO) are frequently used in insurance studies, highway design, and development planning to delineate flood boundaries. The WSPRO model requires input of horizontal and vertical coordinate data that define cross-sectional river-channel geometry. Cross-sectional and other hydraulic data are manually coded into the WSPRO model, a labor-intensive procedure. For each cross section, output from the model assists in approximating the flood boundaries and high-water elevations of floods with specific recurrence intervals (for example, 100-year or 500-year). The flood-boundary locations along a series of cross sections are connected to delineate the flood-prone areas for the selected recurrence intervals.

To expedite the data collection and coding tasks required for modeling, the geographic information system (GIS), ARC/INFO, was used to manipulate and process digital data supplied in AutoCAD drawing interchange file (DXF) format. The DXF files, which were derived from aerial photographs, included 2-foot elevation data along topographic contours with +0.5-foot resolution and the outlines of stream channels. Cross-section lines, located according to standard step-backwater criteria, were digitized across the valleys. A three-dimensional surface was generated from the 2-foot contours by use of the GIS software, and the digitized section lines were overlain on this surface. GIS calculated the intersections of contour lines and cross-section lines, which provided most of the required cross-sectional geometry data for input to the WSPRO model.

Most of the data collection and coding processes were automated, significantly reducing labor costs and human error. In addition, maps at various scales can be easily produced as needed after digitizing the flood-prone areas from the WSPRO model into GIS.

Introduction and Problem Statement

Losses due to flood damage generally cost the American public hundreds of millions of dollars annually. In 1968, the National Flood Insurance Act established the National Flood Insurance Program (NFIP) to help reduce the cost to the public and provide a framework to help reduce future losses. The Federal Emergency Management Agency (FEMA) administers the NFIP. As listed in Mrazik and Kinberg (1), the major objectives of the NFIP are to:

- Make nationwide flood insurance available to all communities subject to periodic flooding.
- Guide future development, where practical, away from flood-prone areas.
- Encourage state and local govemments to make appropriate land use adjustments to restrict development of land that is subject to flood damage.
- Establish a cooperative program involving the federal government and the private insurance industry.
- Encourage lending institutions, as a matter of national policy, to assist in furthering program objectives.
- Authorize the continuing studies of flood hazards.

Studies of flood-prone areas typically involve using step-backwater computer algorithms (digital models) to estimate river water-surface profile elevations and flood-inundation patterns along the topography of the river and its overbanks. FEMA recognizes the U.S. Geological Survey's (USGS's) step-backwater model, Water Surface PROfile (WSPRO), as a suitable computer model for use in flood insurance studies (2, 3). Basic data input for step-backwater models includes:

- Estimates of flood discharge and initial water-surface elevations.
- Stream cross-sectional geometry.
- Roughness coefficients for cross sections.

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- Contracted opening geometry if bridges or culverts are located along the study reach.

Obtaining meaningful model results typically requires numerous stream cross sections referenced to a common elevation datum along a stream reach. The data-collection efforts to obtain these cross-sectional data require costly, labor-intensive fieldwork. Study efforts along lengthy stream reaches may, however, involve the generation of a contour map using aerial photogrammetric mapping techniques. Processing the spatial data may still require extensive labor to extract the cross-sectional data needed for the WSPRO model.

The development of geographic information systems (GIS) technology has greatly enhanced analyses of spatial data such as topography. In an effort to improve the quality of mapping and delineation of flood-prone areas in Summit County, Ohio, the USGS developed a method of using a GIS as a pre- and postprocessor of the input and output data for the WSPRO model. This paper describes the steps the USGS used to develop this interface and discusses some difficulties encountered during the process.

Approach

Several steps were taken that resulted in the delineation of a flood-prone area in Summit County, Ohio. These steps are shown in a flow chart (see Figure 1) and described below.

Data were obtained for this study via aerial photography during April 1990. These data include mappable features at the given scale including topography at 2-foot contour intervals, stream boundaries, roads, and buildings. The data are estimated to be vertically accurate to +0.5 feet. The data were put into AutoCAD and were prepared for delivery to the USGS on 3.5-inch floppy disks in AutoCAD drawing interchange file (DXF) ASCII format. ARC/INFO was used to convert the DXF file into two separate data layers containing only the topography and traces of stream banks within the study area.

A three-dimensional surface was generated from the topographic data using the ARC/INFO software package Triangulated Irregular Network (TIN). Cross-section lines were digitized over the topography data layer. The cross sections were placed according to standard step-backwater criteria (4) and were generally:

- Perpendicular to stream flow
- At major breaks in streambed profiles
- At minimum and maximum cross-sectional areas
- At major changes in stream conveyance
- Spaced about one cross-section width apart

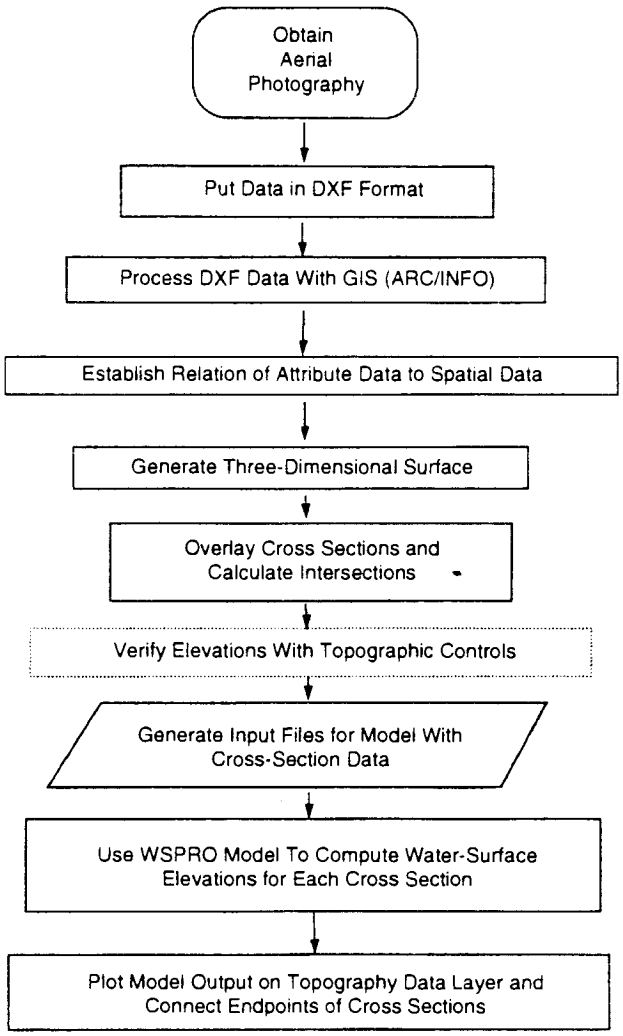

Figure 1. Flowchart of data conversion and processing for use in the Water Surface PROfile.

The cross-section lines were then overlaid on the three-dimensional surface of topography, and GIS calculated the intersections of the contour lines and cross sections. The locations and elevations of these intersections were output as an ASCII file and slightly modified for input into the WSPRO model.

These GIS data were used along with the aforementioned required data as input to the WSPRO model. Input for the model included estimates of the 100-year flood discharge (5), stream cross-sectional geometry (supplied by this work), and estimates of roughness coefficients for cross sections. The WSPRO model was then run, providing output in the form of water-surface elevations at specific distances along section lines corresponding to the simulated elevation of a 100-year flood.

Points corresponding to the flood elevations along the cross-section lines were plotted on the topography data

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layer and were connected manually (to delineate flood boundaries) by interpolating the elevations with respect to adjacent contours. A polygon of the flood "surface" was generated and drawn on a map (see Figure 2).

Results

The supplied topographic data were of sufficient quality and resolution to substitute for field-surveyed elevations; however, field surveys to verify the elevations along the cross sections would augment this quality control process (see Figure 1). Typically, a crew of two individuals may take up to 4 days to survey and reduce the field data for the study area chosen for this study. Because aerial photography is commonly substituted for land surveying, the most significant effort and source of error may come from manually extracting elevations and distances along cross sections for input into the WSPRO

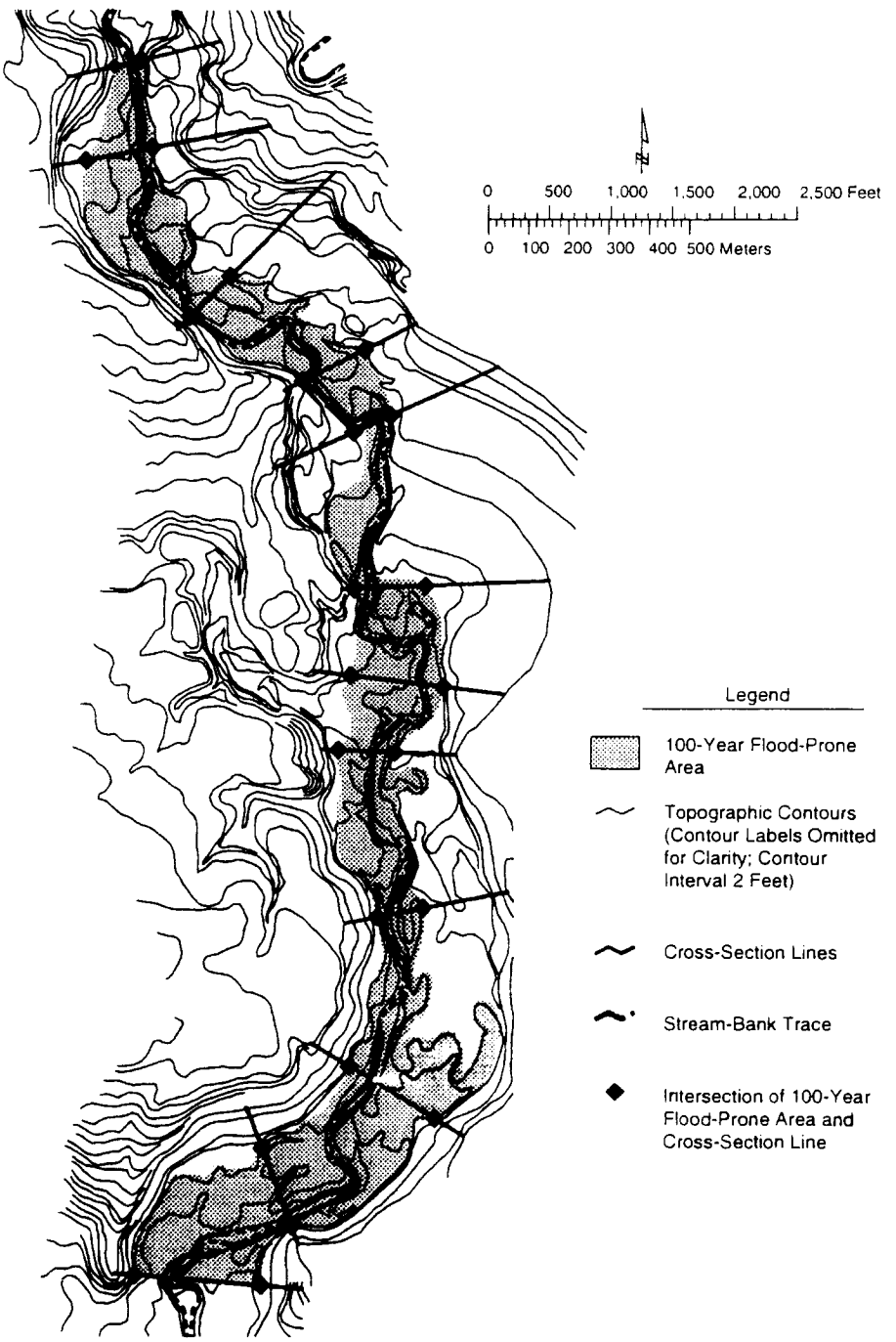

Figure 2. Watershed showing delineation of 100-year flood-prone area.

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model. Initial development of the method to use GIS for this analysis took approximately 1 week to refine; however, future analyses would probably only take one person 1 day to perform. This represents a significant cost savings. Additionally, reducing the amount of human-induced error can substantially improve the reliability and accuracy of the computer-generated flood-prone area data.

Because topography, stream traces, and other features are supplied in the DXF file, these data can easily be brought into GIS. Maps can be made that show these features in relation to the predicted flood-prone area. Maps showing a variety of features can be produced at any scale, with accuracy limited only by the accuracy of the source scale. Additionally, GIS can calculate the intersection of map features that may lie within the flood-prone area, such as buildings that may contain hazardous materials. GIS can also overlay land use data layers within the flood-prone area to define areas that should not be developed or that have already been overdeveloped in accordance with the aforementioned NFIP objectives.

FEMA now requests that future flood-study mapping be completed using GIS format, a common goal that both the USGS and FEMA are working toward. These data are important to land planners, flood-plain regulators, and insurance companies that rely on accurate estimates of flood-prone areas. By increasing the accessibility of the data by using GIS, we can substantially improve our ability to analyze spatial data efficiently.

Problems Encountered

Problems using data supplied in DXF format in conjunction with GIS resulted primarily from the fact that the DXF data were prepared for the purpose of making a topographic map, not a GIS data layer. The contour lines were segmented; that is, where ends of segments met, they were not physically connected to form a topologically viable data layer. The data layer needed to be edited because GIS requires topology for spatial-data processing. Additionally, in areas where the topographic gradient was particularly steep, contour lines were omitted. In both cases, an attempt was made to allow GIS to establish a physical connection of contour lines, but subsequent manual interpolation was also required. This may have introduced error into the data set. If future work requires the use of DXF data, the request for data should specifically state that all topographic contours be continuous.

AutoCAD stores data differently from GIS, so a relationship needed to be established between the data file containing elevations and the data file associated with the lines that make up the topography data layer. Several lines from the DXF file did not have any data associated with them, thus necessitating the addition of contour elevation data by context with the adjacent contours that did have data. This step may also have introduced errors, but quality-control measures to verify the topographic contours and contour elevations could help to minimize these errors.

Output from the WSPRO model is in the form of a series of points along cross sections that were connected by manual interpolation. This step also may introduce some error, but the same process must be performed when not using GIS.

Conclusions

This report documents an example of how GIS can be used to facilitate step-backwater modeling of flood-prone areas. The results of the study show that significant savings may be expected in the form of reduced labor requirements. Furthermore, FEMA now requires the use of GIS to conduct flood-study mapping, thus providing a means to conduct additional spatial analyses more efficiently. As aerial photography and GIS technology improve, although additional sources of error may arise, the overall accuracy, reliability, and reproducibility of the model input and results should also improve.

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Small Is Beautiful: GIS and Small Native American Reservations—Approach, Problems, Pitfalls, and Advantages

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Background

The Lower Sioux Indian Reservation

The Lower Sioux Indian Reservation covers 1,743 acres in southwestern Minnesota bordering the Minnesota River. The land base consists of several hundred acres of prime, flat agricultural land, a large wetlands slough complex, prairie pothole wetlands, bottom land wetlands, small lakes, and approximately 250 acres of timber and brush. The elevation ranges from Minnesota River level to the adjacent bluffs several hundred feet higher.

This rural reservation contains a moderate amount of infrastructure, including paved and dirt roads, 12-acre sewage lagoon serving a moderately sized casino, community water system composed of a tower and small treatment plant for the 90 mostly single-family dwelling homes, convenience store/gas station/gift shop, community center, small two-room schoolhouse, pottery works with gift shop, warehouse, and church. The casino-fueled economic boost to the community recently resulted in improvements to infrastructure and plans for additional projects.

Office of the Environment

The tribal government was formed under the Indian Reorganization Act of 1934. The governing body is an elected five-person tribal community council that administers several government departments and is responsible for all government activities.

Under a U.S. Environmental Protection Agency (EPA) Region 5 multimedia grant, the Upper Sioux and Lower Sioux Office of the Environment (OE) was formed in late 1992. This unique joint venture between two tribes and EPA envisioned moving the tribal governments into compliance with major federal environmental legislation. At the present time, only the Lower Sioux are developing a tribal geographic information system (GIS). Therefore, this article is solely applicable to this community, although adoption of an Upper Sioux Community GIS would likely follow a similar lifeline.

Environmental Regulation in Indian Country

Reservations are subject to a bewildering array of environmental regulations. Numerous meetings, publications, projects, and court decisions are devoted to determining what law does or does not apply on any particular reservation. In very general terms, the following can be stated: state environmental regulations do not apply, federal regulations do apply, and tribal regulations may apply. From a tribal environmental employee's point of view, numerous environmental regulations (whether federal or tribal) do exist that apply to reservation activities and land, and they require compliance.

The Problem and the Solution

The OE's responsibility is to bring the reservation into compliance with the 14 major pieces of environmental legislation administered through EPA and directly applicable to tribes. The OE finds itself responsible for any and all other applicable environmental regulations and all other less-regulated environmental media. The OE currently has a staff of one.

In addition to the responsibility of moving the tribes into compliance with federal environmental regulations, the OE also develops environmental infrastructure, institutes environmental programs, and performs grant writing. Lower Sioux programs currently include Clean Water Act (CWA) and Safe Drinking Water Act (SDWA) compliance, solid waste planning including development and institution of a household recycling program, wetlands regulations compliance, wetlands mapping and restoration, National Environmental Policy Act (NEPA) compliance and site assessments, basic hydrological data gathering and mapping, radon testing and mitigation, environmental education as necessary, SARA

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Title III compliance and planning, and a variety of related tasks.

Contracts or grants are currently being administered under several Bureau of Indian Affairs (BIA) programs, U.S. Geological Survey (USGS) and U.S. Army Corps of Engineers (COE) matching funds programs, two EPA programs, one Federal Emergency Management Agency (FEMA) program, one Administration of Native Americans (ANA) program, one Great Lakes Intertribal Council (GLITC) program, and a cooperative project with the National Tribal Environmental Council (NTEC).

Needless to say, responsibilities of the OE are limited by staff hours rather than need. As distressing as the reservation's unaddressed environmental needs are, equally distressing (prior to GIS development) was the helter skelter manner in which the OE digested the data and information flowing into the office. Because of its broad responsibilities and the administrative problems being encountered, the OE began to investigate developing a tribal GIS.

The Lower Sioux GIS

System Selection

The Lower Sioux GIS system is a networked PC station through the Bureau of Indian Affairs Geographic Data Service Center's (BIA GDSC's) two Sun MP690 SparcServers in Golden, Colorado. The GDSC is the hub of BIA's GIS and remote sensing program, known as the Indian Integrated Resources Information Program (IIRIP). The purpose of the IIRIP is twofold: first, make GIS and remote sensing technology available to tribes and BIA personnel; second, transfer these technologies to tribal organizations.

Database development and management functions, technical support, development of simplified user interfaces, remote sensing interpretation, and implementation of equipment directives are performed by approximately 30 GDSC employees for approximately 230 GDSC users. User technical support is also available through BIA field offices, each of which has a designated GIS coordinator. Simplified user interfaces for specialized programs have been developed including the Lightening Display System and the Land Title Mapping System. Quality control is provided for non-BIA-produced data that will be inputted.

The GDSC has standardized on the ARC/INFO family of software produced by Environmental Systems Research Institute (ESRI). GDSC has developed a number of hardware/software configuration options depending on tribal needs and financial resources and based upon GDSC experience. The OE happily relies on this experience to avoid the familiar horror stories related to equipment and software incompatibility.

Based upon GDSC configuration advice, the initial GIS setup will be on the OE's existing Compaq PC using Tektronix Terminal Emulation software (EM4105) and a Multitech modem (MT932BA). The system can use the OE's Hewlett Packard (HP) Deskjet 500, although a significant upgrade, possibly to an HP Paint or HP Excel Paint, is soon expected. Initial startup hardware and software costs are minimal in this configuration. Costs for the above equipment and introductory training are less than $5,000.

GIS Users at Lower Sioux

Initial setup and data loading will be in the OE, and the OE employee will receive introductory training on the system. Because the OE is formed through a cooperative agreement between two tribes, the Upper Sioux and the Lower Sioux, the OE is centrally located between the reservations. The system will probably be relocated to the Lower Sioux Community Center within 1 year. A tribal government employee will receive advanced GIS training and be available for all tribal government departments and businesses.

Funding

In addition to tribal contributions, funding has come through several sources and joint agreements with the tribe and BIA, EPA, and ANA.

Training

The GDSC supplies no-cost training to tribes. The *Geographic Data Service Center 1995 Training Catalog* (no federal document number available) offers eight formal courses repeatedly throughout the year, a 5-week intern program, and a cooperative student program. Courses are held at the GDSC or by request at BIA field offices and tribal locations.

The GDSC also produces the monthly *The Service Center Review* (ISSN 1073-6190), a helpful compilation of current issues, available resources, system bugs, and other items of interest to GDSC users.

Data Collection and Input

Data collection can be divided into three categories: aerial photography, portable global positioning system (GPS) data, and ARC/INFO export files created under contracted studies.

Aerial Photography

Surface features and topography will be obtained using aerial photography reduced to GIS format, then downloaded to the GDSC. Coverages will consist of 62 categories of features on a scale of 1 inch = 100 feet with 2-foot contour intervals.

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Global Positioning Systems

Use of a portable Trimble, Inc., GPS Pathfinder Pro XL submeter GPS mapping system purchased with assistance from an ANA grant will allow updating of surface features and addition of nonsurface features as necessary. It will also facilitate input of attribute data.

The GPS will also be used during field work by USGS on the Lower Sioux hydrological mapping project to obtain data that otherwise would not be put into ARC/INFO export file for any reason (i.e., it might not be directly related to the project at hand or outside the agreed upon data to be converted to ARC/INFO export file form but nevertheless is of importance to the OE). The alternative is that this type of information never makes it into the GIS and is lost.

ARC/INFO Export Files

Fortunately, most federal agencies that supply funding to tribes for environmental work are well versed in GIS applications and the need for GIS-ready data. The OE now requires all information and mapping to be delivered as an ARC/INFO export file with data registered to a real world coordinate system. Downloading of this data to the GDSC mainframes allows for direct input of the data. The OE has contemplated, but not acted on, conversion of existing data for the GIS. This is an expensive and time consuming process that must be weighed in comparison with recollecting the data. Ironically, the lack of reservation data therefore becomes a benefit because time consuming and expensive data conversion is unnecessary.

The Intertribal GIS Council

Information gathering, networking, and addressing uniquely tribal problems were some of the accomplishments at the first annual meeting of the Intertribal GIS Council (IGC) held in June 1994. Vendors as well as BIA regional office and GDSC representatives answered questions and presented panels. This annual conference is likely to become a major benefit to the tribe as it continually develops the GIS.

The Future

As the tribal government becomes more familiar with the GIS, its uses, and advantages, recognized governmental needs will likely drive the development of further coverages. The OE also expects to access existing governmental data of importance to the tribe in an effort to expand the GIS database and is actively seeking sources of such information.

Philosophical Caveat

Albert Einstein stated that, "The significant problems we face cannot be solved at the same level of thinking we were at when we created them." Some assume that GIS is the next level of reasoning in the environmental profession because we can accomplish tasks more quickly, more efficiently, with more variables accounted for, and beyond what we could have hoped to accomplish prior to GIS.

Essentially, what we have gained is speed and the capacity to include additional data, which is not what Einstein was referring to when he spoke of the next level. Wisdom, in the sense of a higher level of understanding, is the necessary ingredient to the solution of current environmental problems; in other words, movement beyond the paradigm that created the problem. GIS may be the tool that pushes the environmental professional to the next level of wisdom by presenting the data and information in a manner that allows the user to stand back and see more clearly on a higher plane. But that level can be found only within the environmental professional himself or herself and not within GIS.

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A GIS-Based Approach to Characterizing Chemical Compounds in Soil and Modeling of Remedial System Design

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Introduction: The Problem

The cost-effectiveness of implementing a computerized geographic information system (GIS) for environmental subsurface characterization should be based on long-term remedial objectives. A GIS project was developed to characterize soil contamination and to provide design parameters for a soil vapor extraction remedial system, as part of a $120-million remediation and "land sale" project in California. The primary purposes of the GIS were to efficiently combine and evaluate (model) disparate data sets, provide "new" and more useful information to aid in short-term engineering decisions, and support the development of long-term cleanup goals.

The project had a major change in scope early on, and the schedule was expedited to allow for the development of "land sale" options and for actual site redevelopment at the earliest opportunity. Characterization of chemically affected soil would have been compromised given the above circumstances without an ambitious undertaking of concurrently developing and implementing a GIS with three-dimensional (3-D) geostatistical and predictive modeling capabilities.

The GIS Approach

Computer solutions included the use of a cross-platform (DOS and UNIX) GIS to quickly and systematically incorporate spatial and chemical data sets and to provide a distributed data processing and analysis environment (see Figure 1). Networked, DOS-based relational databases were used to compile and disseminate data for the numerous investigatory and engineering tasks. UNIX-based computer aided design (CAD) and modeling applications received data from databases, performed quantitative analyses, and provided 3-D computer graphics. Given the aggressive project schedule, exclusive use of one platform would not be realistic due mainly to the limited data modeling capacity and 3-D graphics in DOS systems. On the other hand, the high startup and operating costs of several UNIX workstations would render their exclusive use much less cost-effective.

The hardware and software configurations were integrated in a client/server Intergraph InterPro 6400 with 48 megabytes of memory. It is largely a 3-D CAD system with add-on modules of geologic mapping and 3-D visual models capable of consolidating both environmental and engineering parameters for analysis (see Figure 1). Textual environmental and geologic data were extracted by SQL queries from relational databases and were transferred to mapping and modeling modules via PC/TCP cross-platform linkage.

The GIS assisted in making short- and long-term decisions regarding health-risk-based regulatory strategy and engineering feasibility. Use of spatial statistical and predictive models was part of a GIS-based decision-making loop (see Figure 2). The process supported concurrent activities in:

- Data collection: field program.
- Numerical models of remedial system configurations.
- Development of cleanup goals from health risk assessments.
- Remedial design with CAD capability.

Site Background

In early 1993, the remedial investigation of the operable unit for soil at a former aircraft manufacturing facility in southern California was thought to be ready for remedial alternatives feasibility study. ICF Kaiser Engineers, Inc., was awarded the contract to perform feasibility studies on applicable soil cleanup technologies and to subsequently design and manage the installation and early operation of the selected technologies. After $700,000 was spent evaluating data collected by previous consultants, it was decided that an additional $5 million worth

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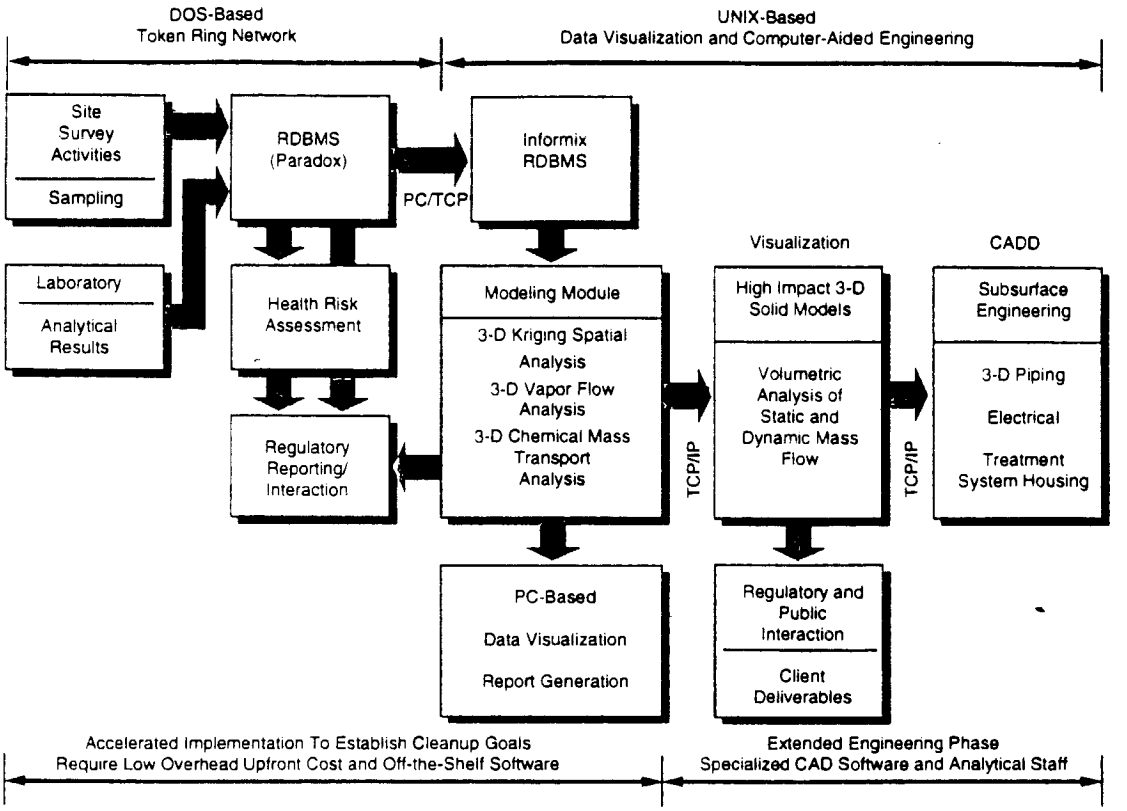

Figure 1. Multiplatform GIS project.

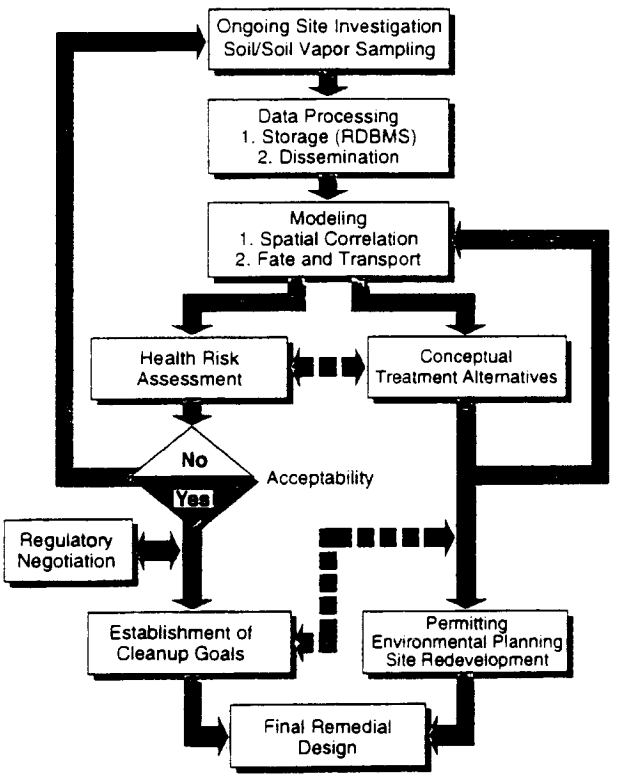

Figure 2. GIS-assisted decision tree.

of field activities were required to more definitively estimate the volume of chemically affected soil and the nature and extent of contamination at the facility. Because of the data gaps, the selection and design of alternatives could not be addressed with a high degree of certainty. Hence, computer assisted data processing was crucial to speed up the feasibility study, accelerate downstream work, and reduce the overall project schedule to the minimum.

The site is environmentally complex, covering an area of approximately 120 acres. As a result of nearly half a century of aircraft production and development, soil beneath the facility is affected by fuel and heavy oil hydrocarbons (TPH) commingled with volatile compounds, mainly perchloroethylene (PCE) and trichloroethylene (TCE) (see Figure 3). Ground water at 170 feet below ground surface is affected by TCE and PCE, but it is not part of the drinking water aquifer. The facility has been demolished, and shallow contaminated soil has been excavated and back-filled to an interim grade.

Methodology

Health-Risk-Based Cleanup Goals

Central to determining the volume and kinds of data to be collected was the question of whether chemicals in soil represented potentially unacceptable risks to human

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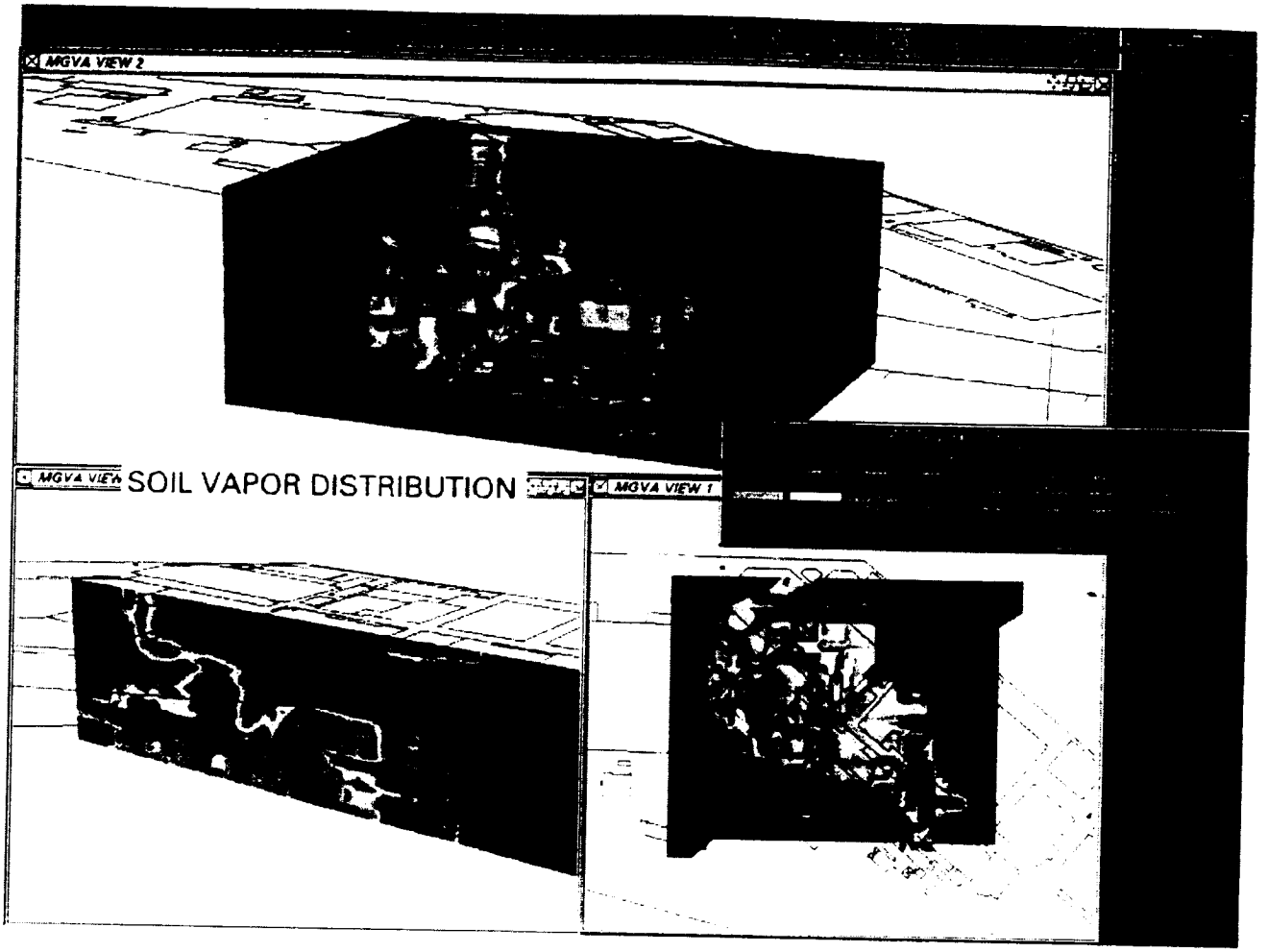

Figure 3. Aircraft manufacturing facility in California. Outline of demolished buildings located at the 120-acre site are shown as surface features for reference. A geostatistical model of a 3-D kriged VOC soil vapor cloud in the subsurface was simulated with Intergraph's MGVA. Views displayed are: 3-D isometric, vertical section of chemical isoplaths, and a nearly plan view. Digital simulation also illustrates VOCs affecting ground water in a dispersive nature at a depth of nearly 170 feet bgs (shown at bottom of isometric view).

health and to the environment, with the former being of particular concern to construction workers onsite during redevelopment.

Because site redevelopment was scheduled to begin in the near term, data collection and GIS analysis concentrated on shallow depths (top 20 feet), with decreasing sample density at greater depths. A health-risk-based cleanup goal (HBCG) approach to collecting more data was to establish cleanup goals for near-term remediation of the shallow soils as well as for long-term remedial measures of contaminated soils at greater depths. Further, various regulatory agencies had to approve the estimated cleanup goals in a short time. Ongoing site demolition and excavation schedules encouraged the aggressive regulatory negotiations. The shallow cleanup goals for volatile organic compounds (VOCs) and TPH determined the volume of contaminated soil to be removed. At greater depths, data gaps were minimized to more definitively characterize the nature of TPH and VOC contaminations and to facilitate the implementation of long-term remedial objectives (i.e., in situ soil vapor extraction).

In situ soil vapor and soil sampling composed the field program, which provided data to map the subsurface distribution of volatile organic compounds, including TCE and PCE. Only in situ soil sampling was used for characterizing TPH. The ratio of soil vapor to soil samples was 4:1. No previous soil vapor information was available. ICF Kaiser has been refining the technique of comparing results from paired soil vapor and soil samples in past and similar projects. Hydraulic probes were used instead of drilling to acquire soil vapor samples at shallow depths. This minimized waste and cost in the field program significantly.

Risk Assessment and Spatial Analysis

Human health risk analyses were conducted for the entire site, and risk factors were contoured and overlaid

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on maps of past usage and known soil contamination areas. Before the risk modeling could proceed, chemical and lithological data gathered in the past 7 years and those acquired by ICF Kaiser populated the environmental relational databases. Approximately 522 soil vapor probes were located in 100-square-foot spacings with additional probes in areas requiring better plume definitions. The database contains approximately 15,000 xyz-records of soil and soil gas laboratory analytical results. This information in text and graphics form, combined with site infrastructures and building outlines with attributes of "past usage," were stored as map layers, making up the GIS nucleus. Accuracy of site maps was verified with aerial photographs when available. Data types combined for computerized evaluation included known locations of contaminated soil, contaminated ground water, soil types, and site features. Composite risk maps of the above data were analyzed for data gaps at discrete depth intervals. This analysis was performed while the field program was in progress and hence gave guidance to optimize the locations of additional data points and to minimize the number of samples taken.

The MGLA/MGLM mapping module and the MSM terrain modeling module tracked the earth excavation and removal of contaminated soil. Excavation was largely part of site demolition. It also expedited the removal of TPH contaminated soils, however, because no other short-term means of remediation are available for these substances. Tracking of removed soils was essential because concurrent field activities were occurring in site demolition, data gathering, and risk modeling.

The GIS coordinated all three. Geologists and surveyors provided terrain data from daily excavation activities, which were transcribed into database formats. Maps illustrated the locations of excavated soil and removed chemicals in soil at various depths. Although TCE and PCE were of foremost concern as health risks, all compounds and some metals identified in soil were screened for unacceptable risk. Terrain modeling (mapping) as part of health risk assessment may seem unusual, but results of estimated cleanup levels and accurate locations of left-in-place contamination, mostly soils at greater depths, were critical to the cost-effectiveness and proper design of long-term remedial systems.

Characterization of Subsurface VOCs

In situ soil vapor extraction (SVE) of total volatile compounds in dense nonaqueous, liquid, gaseous, and adsorbed solid forms in the subsurface produced favorable results that have been well documented in recent years. ICF Kaiser proposed a very large-scale SVE system (see Figure 4), perhaps the largest yet, as long-term remedial technology for this former aircraft manufacturing site. The primary design problem was speculating on air flow capacity and operating time of the complex system components. The SVE system comprises three fundamental elements:

- Front-end, in situ subsurface vents (totaling 193 corings).
- Applied vacuum and air transport manifolds linking the subsurface vents to the treatment compound (distance of one-quarter mile with over 100 manifolds).
- A multivessel activated carbon treatment system.

To size the pipes, carbon vessels, and vacuum required to achieve a certain rate of VOC removal, the total mass and nature of sorption had to be known. Due to the schedule-driven nature of this project, the SVE design accounted for the time needed to accomplish the cleanup goals.

To estimate the extent and total mass of VOCs in the subsurface, soil vapor data were input to a 3-D kriging algorithm (1) to produce a concentration continuum model (see Figure 3). This solid model of predicted total VOC concentrations took the form of a uniformly spaced 3-D grid-block that completely encased the site. Cell sizes ranged from 10 to 20 cubic feet, depending on the model run, number of data clusters, density of data points in areas of clustered data, and the standard deviation of variances for estimated values in all cells. The Fortran program estimated a concentration value for each cell based on the nearest field sample(s).

The validity of such "block kriging" models can be judged by the size of the variances, smoothness, and agreement with nearby field data. Because volume is a known quantity in kriging, the total mass can be calculated by incorporating soil bulk density or porosity, both of which were less than abundant for this investigation. Renderings of kriged results in 2-D plan view contour maps, cross-sectional maps, and 3-D "vapor cloud" (see Figure 3) were included in client reports and used in regulatory presentations and public forums.

Remedial Design Layout

Final Extension of a Fully Integrated GIS

With the total mass and extent of VOCs derived from 3-D kriged results, the applied vacuum at individual vent heads and the cumulative pressure (negative) necessary to extract and transport VOC vapors from the subsurface to the treatment system can be estimated. We performed 3-D air flow analysis by use of finite difference fluid flow models and chemical transport models. The Fortran codes used to approximate compressible flow and chemical transport were AIR3D (2) and VT3D (3), respectively. Air flow simulations focused on maximizing vacuums at the shallow depths down to 20 feet to expedite remediation of contaminated soils that were

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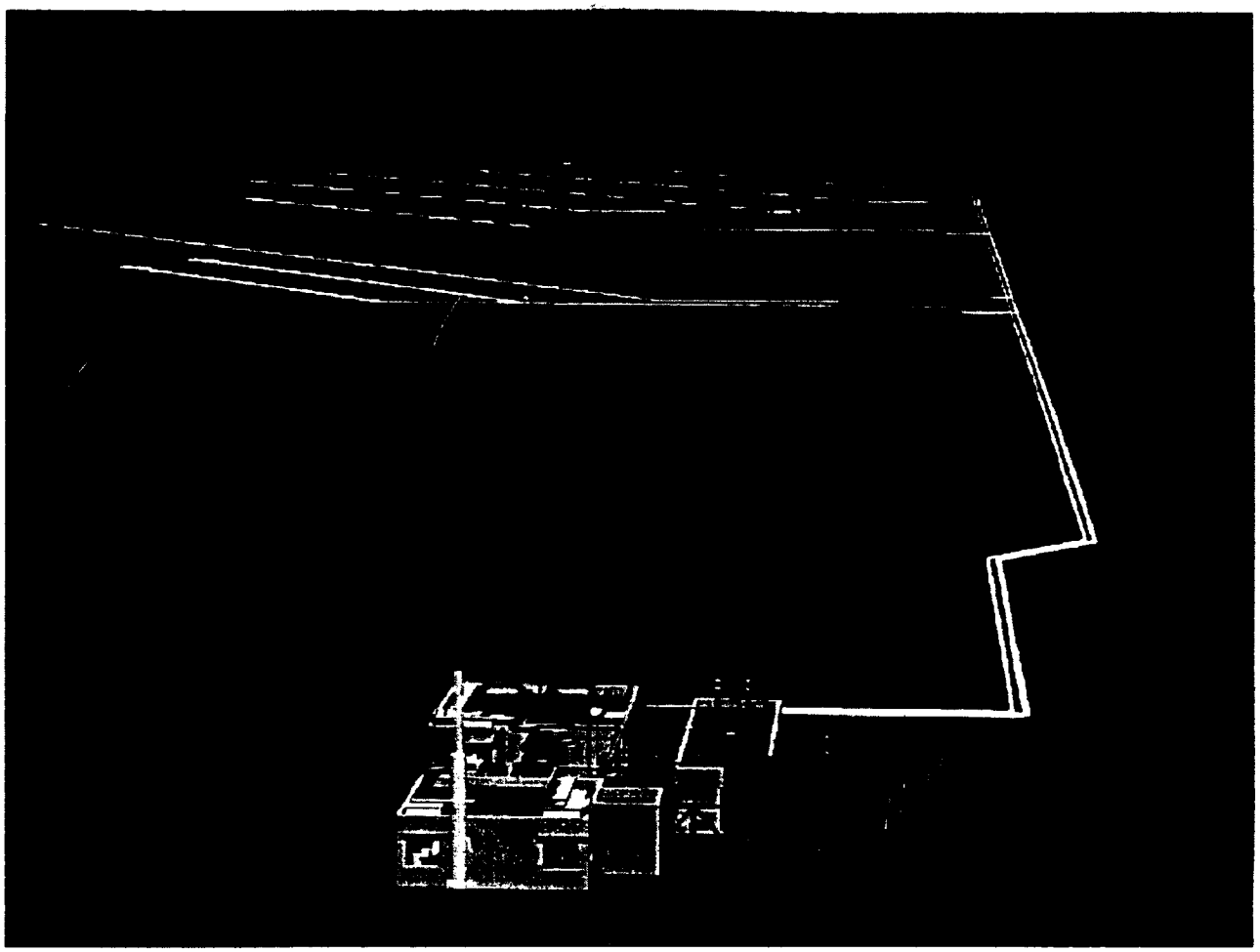

Figure 4. A rendering by the Intergraph 3-D Plant Design System of an in situ soil vapor extraction and treatment system. The cutaway section located near the upper left portion of the figure exposes some of the 193 subsurface extraction vents bottoming at 120 feet (bgs). These vents are located in a cluster for long-term extraction of the VOC vapor cloud presented in Figure 3. Vents are connected to a system of parallel airflow manifolds (right side of figure), which runs one-quarter of a mile to the treatment compound (foreground of figure).

not removed during site demolition and excavation. The lower depths were also included in each simulation. Transient mass transport models incorporate flow fields, given by flow models, and predicted cleanup times based on established HBCG cleanup goals. As VOC concentrations in an operating SVE system fall below cleanup levels in the top 20 feet, thus minimizing human risk, available vacuums thereafter will be diverted to vents at lower depths to be part of long-term extraction scenarios. Models suggested that cleanup for the top 20 feet can be accomplished within 1 year.

Numerical models prescribed vacuum levels at each vent head, which is the aboveground segment of a subsurface SVE vent. The 193 vents are connected to a system of parallel manifolds (see Figure 4) that transport vapor to the treatment system. With the vacuums known at vent heads, the size of manifolds and capacity of vacuum blowers can be determined and integrated into the overall system design. With 3-D Plant Design module as part of the Intergraph CAD/GIS, manifold layouts and treatment compound can be modeled in 3-D and easily checked for pipe routing interferences. The final layout of the SVE system was overlaid onto contour maps of total VOC concentrations to check on accuracy and completeness of vent locations and manifold layouts.

Conclusion

Maximized Visual and Analytical Responses

One goal of this project was to expedite regulatory negotiations and gain early acceptance of cleanup goals. The computerized data processing and visualization contributed generously to the rapid understanding of modeling results by expert regulators and the lay public. Likewise, the GIS facilitated the response to regulatory comments. Positive comments first came

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from the client's in-house review of model results and the high impact 3-D color rendering of kriged VOC distributions in the subsurface (see Figure 3).

Analytically, benefits were derived from the efficiency of electronic data access and the ability to "predict" the presence of contaminant in areas with sparse field data. The process of kriging involves the linear interpolation and extrapolation of existing data. The resultant contaminant distribution is a "conservative" model that provided the best fit with field data and validated conceptualized subsurface conditions. Further, models provided conservative estimates of mass and extent of PCE and TCE contaminations. Kriging also provided information on the uncertainty of the predicted chemical distribution, which is extremely useful for regulatory discussion and system design. The efficiency of computer models allowed investigators to perform numerous model runs with varied boundary parameters, such as cell size and search radii, in the kriging process.

Accurate mapping of excavated soil and the removal of most TPH source areas provided the incentive to critically assess the feasibility of a no-action remedial scenario for these substances at greater depths. With removal of many TPH source areas, 1-D finite difference models (4) were used to assess the mobility of TPH in NAPL and adsorbed residual phase. Specifically, models assessed the likelihood of largely residual-phase TPH affecting ground water and migrating upward to affect indoor air volumes via gaseous diffusion. Results were extremely favorable; models predicted negligible likelihood of TPH affecting ground water or indoor air volumes. Combined with GIS graphic evidence of specific areas of excavated soil and the absence of TPH sources, regulatory agencies accepted the model results, and the no-action remedial alternative for TPH was approved.

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Polygon Development Improvement Techniques for Hazardous Waste Environmental Impact Analysis

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Introduction

Recently, concern has arisen regarding the effect Superfund sites have on surrounding communities and, specifically, the distribution of those impacts on target populations. In designing geographic information systems (GIS) applications for analyzing potential impacts of hazardous wastes or waste sites on adjacent neighborhoods, many challenges may be encountered. GIS database design requires addressing questions of time, space, and scale.

The U.S. Environmental Protection Agency (EPA) and other federal agencies have conducted studies that indicate that certain sectors of the population may be more vulnerable to exposure to toxics than others. To date, federal departments have enlisted in several GIS-based research projects that attempt to delineate "geographic hot spots" of toxic contamination. Such GIS applications at hazardous waste sites have typically used polygons to represent data from census tracts and/or municipal boundaries. In most cases, however, census tract and other boundaries do not necessarily jibe with community and neighborhood boundaries; therefore, the polygons representing characteristic data for target populations may not be consistent with the actual status of those populations.

The objective of this paper is to demonstrate GIS methods for producing, to the greatest degree possible, socioeconomically and culturally homogenous polygons for impact analysis of specific sensitive populations and/or communities. The paper presents case studies of community/neighborhood characterization problems encountered in developing polygons during previous field investigations involving lead (Pb) contamination, toxic release inventory (TRI) sites, and solid/hazardous waste sites. The paper attempts to demonstrate effective solutions and suggestions for improving polygon development, including GIS data manipulations and software applications. In addition, the paper provides geographic and groundtruthing field methods to support and enhance the accuracy of remotely obtained information. Finally, the discussion includes community and geographic hot spot analyses for potential public health impacts.

Background

In 1992, EPA established the Environmental Equity Workgroup. Its members included personnel from the Offices of Toxic Substances and Civil Rights, as well as Policy, Planning and Evaluation. The workgroup conducted an extensive study on environmental equity issues. Their report offered several recommendations for improving federal agency efforts in protecting minority and low-income populations and recognized a need for more spatial and demographic data. The final report, titled *Environmental Equity: Reducing Risk for All Communities* (1), was released in February 1992 and concluded that "there is (sic) limited data on environmental health effects by race; there are differences by race and income in potential and actual exposures to some pollutants." In response to the above findings, the workgroup offered the following recommendations (1):

> EPA should establish and maintain information which provides an objective basis for assessment of risks by income and race, commencing with developing a research and data collection plan.
>
> It (EPA) should revise its risk assessment procedures to ensure . . . better characterization of risk across population, communities or geographic areas. In some cases it may be important to know whether there are any population groups at disproportionately high risk.

The Agency for Toxic Substances and Disease Registry (ATSDR) formed a community health branch to specifically examine the potential health impact of hazardous waste sites upon people living in surrounding communities. The new branch's personnel direct ATSDR's minor-

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ity health initiative, which focuses upon health threats to minority populations, including those from environmental contaminants.

In addition, EPA established the Office of Environmental Equity. The office's mission includes analyzing environmental impacts upon minority populations, providing technical assistance to disadvantaged communities, and establishing environmental initiatives at minority academic institutions (MAIs). The office serves as a clearinghouse of environmental data and information for groups and individuals involved in environmental equity activities.

In 1993, Representative John Lewis (Democrat-Georgia) introduced the Environmental Justice Act to Congress. The act requires EPA and the Department of Health and Human Services (DHHS) to establish the geographic units for determining environmental high-impact areas (EHIAs), which are the 100 geographic areas found to have the highest volumes of toxic chemical releases.

GIS Applications

GIS could potentially help address the above data and information needs of EPA. The Agency specifically acknowledges this in other recommendations (1):

> EPA could further develop its enforcement prioritization policy to target high-risk populations. Under this scheme, the most exposed and highly susceptible populations in each region would be targeted for enforcement actions. Geographic Information System technology could be used to identify high-risk populations.

Several recent environmental studies have employed computer applications and spatial data. Goldman (2) used GIS in a major study that graphically displayed counties having high percentages of African-Americans, hazardous wastes, and diseases. Mohai and Bryant (3) applied a linear regression model to show a positive correlation between increasing proportions of minority populations and decreasing distances from hazardous waste sites in Detroit. Lavalle and Coyle (4) conducted an extensive analysis of computer databases that hold hazardous waste law enforcement information for the past 10 years. They found inequity in enforcement and remedial actions in white communities versus nonwhite communities.

EPA has enlisted GIS for community environmental impact projects at Regions II and III. EPA's Office of Health Research (OHR) is investigating methods for linking demographic data with TRI information to evaluate the relationship between levels of hazardous waste releases and exposure risks in minority communities. EPA has also developed the TRI "risk screening" process, which employs TRI, U.S. Census data, and GIS to identify TRI releases that may pose significant risk to human health or the environment (5).

Both EPA (6) and the North Carolina Department of Environment, Health, and Natural Resources (7) have recently completed GIS-based environmental investigations. The EPA study involved GIS analyses of TRI chemical releases in the southeastern United States. The report included numerous GIS-produced maps that show locations where TRI releases may be affecting densely populated areas and sensitive ecosystems. The North Carolina study applied GIS in searching for sources of lead-poisoning in children. Findings indicated a positive spatial correlation between high lead-contamination risk communities and those having certain socioeconomic characteristics, such as low income, above-average African-American population percentages, and above-average percentages of residents receiving public assistance.

The ATSDR recently implemented a study using GIS to evaluate and analyze the demographic characteristics of populations near National Priorities List (NPL) sites. According to the ATSDR, "As a result of our pilot tests, we have determined GIS to be the best methodology for identifying potentially impacted minority populations" (8).

Limitations of GIS

While GIS may be a viable tool for investigating environmental inequity, it is not an absolute solution. Issues involving hazardous waste impact assessments tend to be very complex without the added dimension of racism or discrimination. Efforts to determine a causal relationship between the presence of minority communities and environmental hazards must consider the questions of time, scale, and place. Unfortunately, in many instances, GIS applications may be unable to adequately illustrate these three pertinent issues resulting in skewed or altogether incorrect conclusions.

With respect to scale, among the immediate concerns when applying GIS is selecting appropriate sizes for polygons. As indicated above, EPA is in the process of determining the scale for EHIAs. A polygon may be a county or a census tract. Figure 1 illustrates problems

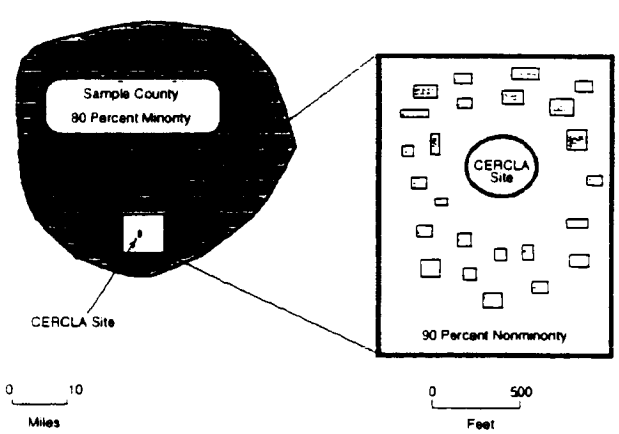

Figure 1. Problems of scale in GIS polygon design.

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of scale associated with polygon size selection. At the county-size scale, a case of environmental inequity apparently exists with the presence of a Superfund site in the sample county because 80 percent of the county's population belongs to a minority ethnic group. A closer look, however, reveals that the population residing in the immediate vicinity of the waste site is predominantly nonminority.

Useful as GIS may be, its output in some cases may display static conditions without considering human movements over time. Figure 2 displays a common situation associated with the "filtering" phenomenon, in which a nonminority population moves out of an area while increasing numbers of an ethnic minority group moves into it. The figure shows that in 1950, a nonminority community surrounded a TRI site (i.e., an active industrial site releasing toxic substances). By 1990, the demographics of the neighborhood had changed along with the status of the TRI site, which is now an abandoned Superfund site. A GIS database probably would contain only information on the community from 1990. Such an instance could suggest that some form of environmental injustice exists given the presence of the Superfund site within the minority community. Accounting for the dynamics of time and human movement, however, would show that the waste site preceded the minority population and that, in actuality, the minority community moved toward the site. This conflicts with the prior notion that unsavory forces placed the site in the minority community.

Figure 3, a schematic of polygons used in an investigation into sources of lead-poisoning in children, also displays the limitations of GIS with respect to time and human dynamics, but at a lesser time interval. The study area is divided into low- and high-risk areas for lead contamination. The locations at which children with lead poisoning were found, however, do not correspond with the areal risk factors. In this case, the GIS is limited in its ability to follow human movements on a daily basis. For instance, a parent with a child who exhibits unhealthy blood-lead concentrations may report the child's home address as someplace within the low-risk polygon. The child may attend school in the high-risk area, however. The children's points of contact with lead may not necessarily correspond with their home addresses, resulting in an inaccurate graphic display.

With respect to place, GIS may be limited in its ability to determine the specific borders of a socioeconomically and demographically homogenous human population. Tosta (9) and Coombes et al. (10) discuss the dilemmas associated with neighborhood boundary delineation in GIS applications. Figure 4 displays a schematic of a census tract. The GIS database may list the tract's per

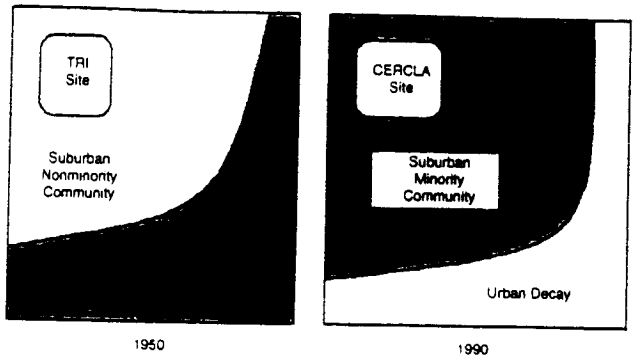
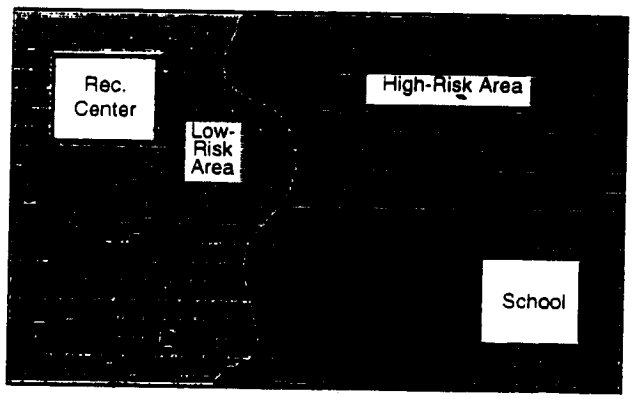
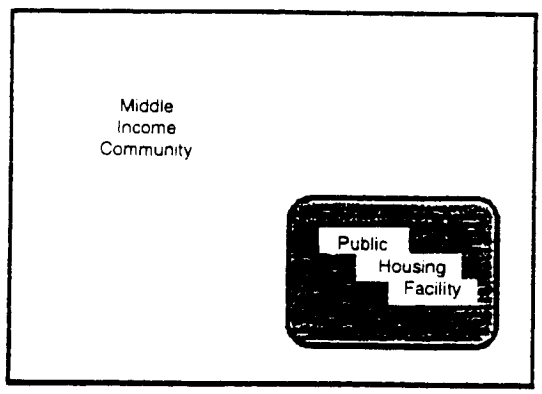


Figure 2. Example of changing community demographics with time near a hazardous waste facility.



● = Homes of Lead-Poisoned Children

Figure 3. Lack of correspondence between locations of lead-poisoned children and high-contamination risk areas due to daily dynamics of human movements.



Census Tract Schematic

Figure 4. Example of significant neighborhood-type variation within a single polygon, possibly resulting in skewed socioeconomic data.

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capita income as relatively low and may list the tract as a low-income neighborhood. Further investigation may find, however, that two very different socioeconomic communities exist within the tract, one middle-class and the other a public housing facility. Frequently, middle-income, African-American communities have low-income housing projects built adjacent to them. With respect to the polygon in Figure 4, if an investigator wanted to research health impacts of toxic wastes for low-income households, using this polygon and others like it would inaccurately depict communities within them.

An additional problem in community definition is determining exactly what defines a minority community. The most common indicator for discerning a minority community would be the existence of a clear majority of some minority group as in Polygon A of Figure 5, or where the minority group makes up more than 50 percent of the population as in Polygon B. In some instances, however, communities have received minority status without the presence of the conditions in Polygons A and B.

Previous investigations show a number of measures used to identify minority communities and census tracts. Greenpeace conducted a 1990 environmental justice study that determined a community's status as minority based upon the relationship between a community's percentage of minority population and the selected minority group's national percentage (11).

Polygon C in Figure 5 depicts Greenpeace's minority community definition. Taking the target ethnic group in Polygon C as African-Americans and the hypothetical extent of Polygon C as the United States (African-Americans make up approximately 12 percent of the total U.S. population), one may determine that Subpolygon C is a minority polygon or community because its minority percentage is over twice that of the national percentage or extent of the large population in Polygon C. The condition that Polygon C illustrates is also evident in a study

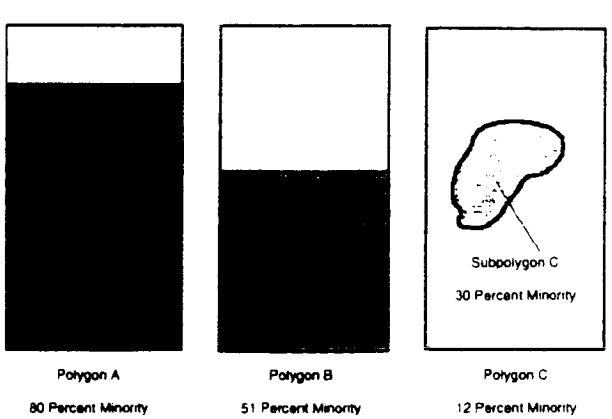

Figure 5. Problems in defining minority polygons.

by Mohai and Bryant (3). The authors claim that environmental inequity exists in Detroit where they found that on average, within a 1-mile radius of the city's hazardous waste facilities, 48 percent of the population is nonwhite.

Solutions With GIS and Supporting Technological Methods

From the above discussion, GIS may appear very limited for use in environmental community impact investigations, but GIS can actually be an extremely effective tool if employed with appropriate supporting technology.

Preliminary Groundtruthing

To design GIS databases that reflect the true nature of target groups and human dynamics, preliminary groundtruthing may be necessary. In some cases, investigators make gross interpretations of suspected environmental inequity without actually visiting the study area. Without groundtruthing prior to final database development, questions of time, scale, and human dynamics may be left unanswered. The integrity of databases produced this way and the antecedent conclusions based upon them may fall into question. Thus, because the nature of environmental and human health impact studies is complex, some on-the-ground work should precede or at least accompany database construction efforts.

Cause-Effect Analyses

Epidemiological studies are increasingly employing GIS. This use is important with respect to environmental investigations because in many cases, proof of a correlation between a waste site and community health problems may be necessary. Croner et al. (12) describe statistics-supported GIS applications for linking "cancer hot spots" with pollution sources. Without conclusive evidence that waste sites and other environmental hazards negatively affect health in socioeconomically disadvantaged populations, claims of environmental injustice may be difficult if not impossible to prove.

In historical analyses of waste facility sitings, GIS may be useful, along with the support of gravity models, in investigating whether the sitings followed the prescribed logic for such siting decisions. Noble (13) wrote that the costliest aspect of waste facilities management is transportation; therefore, siting decisions should favor locations in closest proximity to a selected community's centroid of waste production. Using a GIS gravity model with data for household wastes produced within a given locality, the center of gravity of the volumes of wastes produced could be located. Historical analyses of past siting decisions may find that past siting decisions defied logic. Instead of finding waste sites placed in environmentally safe locations as close as possible to areas of greatest refuse generation, analyses may find instead that sites have been placed farther away in disadvantaged

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communities. Both the taxpaying and potentially affected residents would pay for such unsavory siting practices.

Conclusion

The potential for technological applications, including GIS, in this arena is great. Increased involvement by technologically trained environmental professionals is imminent. Their future involvement, however, must focus on the scientific soundness of investigative methods, data integrity, and the equitable participation of potentially affected citizens in any subsequent decision-making processes.

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Comparing Experiences in the British and U.S Virgin Islands in Implementing GIS for Environmental Problem-Solving

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The British and U.S. Virgin Islands: Comparisons and Contrasts

British Virgin Islands

Three miles to the north and east of the U.S. Virgin Islands (USVI) lie the British Virgin Islands (BVI), a group of 36 islands and cays with a total area of 60 square miles. The four largest are Tortola (24 square miles), Anegada (14 square miles), Virgin Gorda (8.5 square miles), and Jost Van Dyke (4.5 square miles).

Geologically, the BVI belong to the Greater Antilles, and like the USVI and Puerto Rico, rise from the Virgin Banks (or Puerto Rican Shelf). Rocks of the BVI, except Anegada, consist of thick, steeply inclined, metamorphosed volcanic and sedimentary stratified series of Cretaceous age, with dioritic and pegamitic intrusions. Anegada is a 30-foot-high emergent coral limestone platform, apparently from the Pleistocene age.

The BVI are a crown colony of the United Kingdom (UK) with a total population in 1991 of 16,108. Most of the population resides on Tortola (13,225 inhabitants). Between 1980 and 1991, population increased 46.6 percent. The BVI have internal self-government with an elected council headed by a chief minister. The UK appoints a governor to represent the queen and to manage defense, internal security, external affairs, civil service, and court administration.

The BVI economy is based mainly on tourism and servicing international business. Sailing and diving are pivotal features in BVI tourism. The average number of tourists per capita in the BVI is 221, compared with 119 for the USVI (1). The BVI 1990 per capita income was recorded at $10,125. Prior to the 1970s, the BVI economy was based on subsistence agriculture and remittances from British Virgin Islanders who worked in the USVI.

U.S. Virgin Islands

The USVI are an unincorporated territory of the United States, purchased from Denmark in 1917. The total population in 1990 was about 101,000, divided among St. Thomas (48,000), St. Croix (50,000), and St. John (3,500). The tourism-dominated economy of the Virgin Islands generated a per capita Gross Territorial Product in 1990 of approximately $11,000—the highest in the Caribbean (2).

Geographically, geologically, and topographically, St. Thomas (28 square miles) and St. John (20 square miles) are similar; they are both largely volcanic, have deeply indented coastlines, and lie on the Puerto Rican Shelf. St. Thomas and St. John are close to the BVI. St. Croix is a relatively large (84 square miles) and mostly limestone island that lies on its own submarine ridge, which rises more than 4,000 feet from the bottom of the Caribbean Sea. St. Thomas and St. John are about 5 miles apart, and St. Croix is 48 miles south of them.

During the height of tourism development (from the late 1950s through the mid-1970s), the USVI experienced average annual compound population growth rates of over 6 percent, as well as a doubling in real incomes. This unprecedented paroxysm of growth is still being assimilated by a population that differs greatly from the 30,000 people who lived in a predominantly agricultural USVI in 1950. In 1990, the USVI received a daily average of 37 visitors per square kilometer. This compares with a visitor load of 23 visitors per day per square kilometer in the BVI, which is also a high-density tourist destination (1).

Background to GIS Implementation Activities

British Virgin Islands

The idea of geographic information systems (GIS) applications in the BVI first arose with a presentation about

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a proposed project for St. Lucia, made by Dr. Jan Vermeiren of the Organization of American States at the Caribbean Conference of Planners in Kingston, Jamaica, in 1984 (3). The Town and Country Planning Department recognized that it could use GISs analytical and display properties to make presentations to the chief minister and the BVI Executive Council. This proposal fell on fertile ground, given a relatively long-held tradition of support for the Town and Country Planning Department by the United Nations Development Programme (UNDP) and the British Development Division, dating back to the early 1970s.

Subsequent to this inspiration, the Town and Country Planning Department requested budget authority to develop a land use database. This database would include buildings, property boundaries, and constructed and natural features of importance. The Finance Department hoped this project would help combat growing competition to the postal services by independent package delivery services. They renamed the project the National Addressing System, and the legislature provided $200,000 to provide a physical address for each property in the territory.

The Town and Country Planning Department conducted a pilot project, focusing on Road Town, the capital. The pilot project demonstrated that the hard copy land ownership or cadastral maps that the Survey Department was then using were inadequate for accurately accounting for properties, even in the BVI's most developed urban areas. Therefore, the Town and Country Planning Department expanded the scope of the National Addressing System project to identify options for increasing the accuracy of property ownership records, including maps.

Because the Town and Country Planning Department had little experience with computerized land information systems, departmental managers sought support from the UNDP office in Barbados. The UNDP had previously assisted with the department's development control applications database. This mode of operation, in which the BVI purchase services provided through the UNDP, which acts as a "vetting agent" for consultants and other technical assistance, continues to this day. An expert from the United Nations Community and Housing Services (UNCHS) Nairobi office provided the first such consultation by exploring how the BVI might implement a GIS. Over the next few years, three other experts provided input, which the Town and Country Planning Department gradually integrated into a picture of how to use a GIS within the technical and financial limitations of a small island government.

In the meantime, external conditions were improving the chances for the success of the BVI program. British Virgin Islanders were receiving formal and informal training in computer applications in general, and specifically in AutoCAD drafting systems, which increasing numbers of local architects and engineers are using. In addition, the power of rugged microcomputer systems that could withstand the harsh operating conditions of the Virgin Islands was also improving, and local dealers were increasing their skills in support of such systems.

U.S. Virgin Islands

In 1988, a proposal to develop a locally supported GIS was being discussed in detail in the USVI (4). This project resulted in a formal application from the government of the USVI for financial assistance from technical assistance funds provided by the Office of Territorial and Insular Affairs of the U.S. Department of the Interior. The grant was awarded in March 1991. The proposed project combined existing information from several USVI government agencies to produce GIS overlays, as shown in Table 1 (5).

In addition, according to the grant application, (which was written by the Virgin Islands government and may not have represented U.S. Geological Survey's [USGS's] intentions) the National Mapping Division (NMD) of the USGS agreed to digitize the eight USGS 1:24,000-scale quad sheets ("quadrangles") that cover the USVI, including the following categories:

- Roads and trails
  - Power transmission lines
- Hydrography
  - Stream networks
  - Shorelines
  - Wetlands
  - Mangroves
  - Reefs

Table 1. GIS Overlays

Agency	GIS Overlays
DPNR	Zoning
	Flood plain
	Subdivision
WAPA	Water distribution
	Aquifer profiles
	Electrical distribution
VITEMA and emergency services	Critical routes
	Critical facilities
DPW	Sewer line network
	Transportation
	Flood plain

DPNR = Department of Planning and Natural Resources
WAPA = [Virgin Islands] Water and Power Authority
VITEMA = Virgin Islands Emergency Management Agency
DPW = Department of Public Works

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- Topographic contours
- Political boundaries

(These categories were subsequently adjusted based on discussion between the USGS, WAPA, and government agencies. Documentation for these new coverages is not available, but the general idea holds, although no VITEMA data were used, and the USGS did not digitize the WAPA power system.)

The underlying notion was that the whole would be bigger than the parts—each organization would bring its own corporate data and maps so all could share in the digitized product.

Five objectives were identified for the project:

- Provide the USGS in St. Thomas with a complete microcomputer (GIS) workstation.
- Develop a digitized database from USGS topographic maps.
- Contract for the digitizing.
- Acquire digital data to load into GIS.
- Enter data not in digital format.

The proposal required a 50-percent local cost contribution to the project, including a matching suite of hardware for data maintenance, backup, and analysis. The application stated that itemized costs of $305,000 "will be provided by the Virgin Islands Water and Power Authority (WAPA) and the Office of Territorial and International Affairs."

The project application is ambiguous about the nature of the hardware and operating systems that the GIS requires. A list of hardware for USGS use refers to a "microcomputer workstation." Later references to "ESRI ARC/INFO workstation software," however, indicate to sophisticated users that these applications require UNIX workstations and UNIX language operating systems. In general, UNIX is not used or supported in the Virgin Islands. Some Virgin Island officials associated with the GIS project feel they were not fully informed about the GIS operating environment and the long-term support costs to which they were committing.

A frustration for USGS in the project stems from the somewhat limited role the agency has had in providing high-quality data conversion services (digitizing). One USGS staff member explained that unfortunately the agency's mandate only extends to providing data; the agency "can't get involved in applications."

The initial project proposal also was unclear about ownership of the digitized data. The proposal spoke in general terms: "All available digital data and attributes will be complete, accurate, and up-to-date at the end of the project, and will be available for use and transfer to the Department of Planning and Natural Resources (DPNR) (5)."

Given the conditions and environmental constraints discussed, a number of specific differences developed between the GIS implementation processes of the BVI and the USVI.

Initial System Planning Activities

British Virgin Islands

Although the BVI had no formal systems plan, consultants worked with the Town and Country Planning Department on four different occasions to provide insight into some aspect of GIS applications. Sometimes, the benefits from the consultations were neither the type nor the quality the department originally expected. In general, however, each provided some additional perspective on the possible benefits and perils of implementing a GIS.

U.S. Virgin Islands

The USVI apparently made few initial system planning efforts, although the U.S. Department of the Interior and USGS have wide experience in the territory. (An informed source claims a grant was made, possible by EPA, for a preceding $50,000 GIS project, but no mention of this has been found in the materials available for this article, either as a proposal, or in terms of specific products.) Possibly, this very familiarity led to a series of unexamined assumptions and diminished communications about the exact terms of the assistance and services that the USGS exchanged with several agencies of the Virgin Island government.

One indicator of the lack of system planning activities in the Virgin Islands is a proposal that the Virgin Islands Emergency Management Agency (VITEMA) circulated for a "Geographical Information Systems: Technical Operators Meeting." This proposal, from an agency that has always been one of the most important participants in GIS activities, called for a "technical working group" to examine existing database management systems in the territory to develop a planning strategy for implementing GIS.[1] This proposal was dated September 28, 1992, 10 days before the USGS announced a demonstration of the completed products of Phase I of the "comprehensive geographic information system being developed for the USVI (6)."

[1] Ward, R.G. 1992. Geographical information systems: Technical operators meeting. Memorandum to Cyrille Singleton. VITEMA, St. Thomas, U.S. Virgin Islands (September).

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Software Selected and Rationale

British Virgin Islands

In part because of the extended timeframe for the BVI planning and initiation of the GIS system, the Town and Country Planning Department never committed to a specific system configuration until the last stages of the planning process.

This process of "creative procrastination" had three synergistic results:

- PC power increased (and prices decreased) to the point where reasonably priced systems could perform many of the compute- and data-intensive operations demanded for graphics software mapping.
- ARC/INFO released the ARC/CAD version of its GIS software, which worked on PCs within the well-known AutoCAD drafting software. Architect and engineering offices in the BVI already used computer-aided design (CAD) software, so upgrading to include GIS functionality was relatively easy.
- The fourth GIS consultant to work in the BVI was experienced in implementing systems in the Caribbean and had special knowledge and access to early versions of both ARC/CAD and Version 1.0 of ARC/VIEW. These two tools, based on a last minute proposal by International Development Advisory Services (IDAS) of Miami, Florida, a private GIS support contractor, became the basis for the BVI GIS.

U.S. Virgin Islands

Workstation ARC/INFO 6.1 was selected as the basic software for the USVI GIS project because it is the USGS standard software. In the environment surrounding the USVI project, this seemed to be a sufficient explanation, although there may have been other reasons. Because of this decision, however, WAPA and the DPNR lack any means of updating map or attribute data files. Outside providers, such as the USGS's NMD, must perform that service. The USGS has noted that the only reason WAPA and DPNR lack these capabilities is because they (WAPA and DPNR) failed to provide the matching suite of hardware and software specified in the grant application.

The digitized water supply system offers an example of the extra costs that such a condition creates. The USGS built an ARC/INFO coverage by converting mapped data from AutoCAD source files, which they then linked to detailed attribute information about each element (e.g., pipe, valve, elbow) in the system. The USGS then used ARC/INFO network software, purchased with project funds, to build a network model that analyzes and displays the operation of the entire water distribution system—but no software in the Virgin Islands can run the network model.

Hardware Platforms

British Virgin Islands

The BVI GIS was originally installed on a Compaq 486-50, with a 21-inch screen. The Town and Country Planning Department soon learned, however, that data input would be more efficient if two or three smaller machines split the work, with the Compaq available for analysis and data quality checking. The department upgraded its existing office computers to handle the data entry. Users already feel the need for networked applications to share data more quickly. Plans for GIS expansion to other offices, such as the Electricity Corporation, increase the pressure for an extended local area network.

U.S. Virgin Islands

The system that the USGS used to build the USVI GIS database was a Data General UNIX workstation with one large digitizing tablet and one pen plotter. No matching or comparable hardware are installed anywhere in the USVI, as the original project proposal had foreseen. Observers tend to agree, however, that the failure to provide a specific hardware configuration is less significant than the lack of committed, senior, full-time technical staff. This staff is required to operate the level of GIS facility that the USGS envisioned.

Base Map Priorities and Layers Constructed

British Virgin Islands

Building a map database is proving to be a long process for the Town and Country Planning Department. This is complicated by the failure of a key digitizing contractor in Texas to provide property lines in a format conducive to constructing accurate property polygons. Operators in the Town and Country Planning Department have increased their data entry efficiencies, however, and most properties on the most densely inhabited islands have now been digitized.

Producing demonstration data displays accounts for a significant part of the cost of developing databases for the early phases of the BVI GIS implementation. These demonstrations aim to illustrate possible new application areas for other agencies and departments of the BVI that are interested in cooperating and sharing costs of additional system development. For example, the Electricity Corporation and the National Disaster Preparedness Agency need to map emergency services.

Converting the data (i.e., digitizing) in house in the BVI has produced costs and benefits. The costs revolve

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around the steep learning curve for data entry procedures and the constant distractions of responding quickly to "outsiders" who may be important long-term supporters of the GIS. The benefits include increasing staff skills and the ability to build constituencies for the program by promptly responding to real needs.

Coverage priorities for the BVI GIS include a national addressing system, completion of the territorial land use mapping, and accurate cadastral mapping (which has major environmental planning and management implications).

U.S. Virgin Islands

The USGS produced 44 coverages for the Virgin Islands from a variety of sources. Table 2 shows the major coverages and scales, by island.

Table 2. Major Coverages and Scales for the USVI

St. Croix	St. John	St. Thomas
STC water distribution 1:2,400	STJ water distribution 1:2,400	STT water distribution 1:2,400
STC roads 1:2,400	STJ roads 1:2,400	STT roads 1:2,400
STC building footprints 1:2,400	STJ building footprints 1:2,400	STT building footprints 1:2,400
STC shorelines 1:2,400	STJ shorelines 1:2,400	STT shorelines 1:2,400
STC DLG boundaries 1:24,000	STJ DLG boundaries 1:24,000	STT DLG boundaries 1:24,000
STC DLG roads 1:24,000	STC DLG roads 1:24,000	STT DLG roads 1:24,000
STC DLG hydrography 1:24,000	STJ DLG hydrography 1:24,000	STT DLG hydrography 1:24,000
STC DLG hypsography 1:24,000	STJ DLG hypsography 1:24,000	STT DLG hypsography 1:24,000

In addition, the following National Park Service data were converted and added to the data set but were not produced by the USGS:

- STJ NPS boundaries, 1:24,000
- STJ NPS roads, 1:24,000
- STJ NPS hydrography, 1:24,000
- STJ NPS hypsography, 1:24,000
- STJ NPS benthic communities, 1:24,000
- STJ NPS historical sites, 1:24,000
- STJ NPS vegetative cover, 1:24,000

Mapping for St. Thomas, St. John, and St. Croix identified a total of 10 coverages for each island. They are based on information from the USGS ("quad sheets" specifically demarking political boundaries, shorelines and streams, topography, and roads) and higher precision WAPA mapping, which derives from 1986 aerial photogrammetry, including left and right road boundaries, building footprints, shorelines, and water supply system data. The WAPA data are at 1:2,400 scale, an order of magnitude more precise than the USGS base map. St. John mapping consists of 10 added layers based on data that the Virgin Islands National Park (VINP) provided.

The original USVI project proposal referred to a two-phase process of database development, shown below (5):

Phase	Tasks
I. Base system development	Water distribution network Power distribution network Flood plain maps
II. Agency extensions (i.e., by USVI agencies)	Land use maps Transportation networks Emergency facility networks Tax parcel/land value

The USGS announced that Phase I was completed in October 1992 (6). Supposedly, the contents of these two phases were subsequently adjusted to reflect a different range of coverages, but the notion of a "Phase II" in which local agencies would assume more operating responsibilities was retained.

Ownership of, access to, and terms that govern the use of this digital data are confused. The USGS says it is unable to provide an authoritative catalog of the coverages because "one has not been produced." WAPA says it has several diskettes of data in the safe but no equipment to manipulate them. The VINP has learned that it can use its own data as well as WAPA data converted and attributed by the USGS, but the park does not possess or use USGS digital line graph (DLG) data. To personnel in USVI agencies, USGS statements have clouded the question of access to the GIS information. For example, one such statement announced that the USGS Water Management Division cannot make the digitized data available to Virgin Island government agencies.

The DPNR apparently has no means of making direct use of the digital data. First, DPNR has no hardware or software that can use the data. Secondly, it has no operators who can build the GIS systems to actually apply the data to decision-making needs. The department is said to be preparing a new GIS proposal for training, hardware, and software for a new GIS system. According to unconfirmed rumors, this system will be based on MapGrafix, a Macintosh mapping system.

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In operational terms, the data seem to belong to WAPA, which contributed major financial support and map resources to the project. WAPA has been helpful in providing copies of the digital data to other groups and agencies.

Environmental Problem-Solving in Local Decision-Making

British Virgin Islands

In the BVI, the first priority of the GIS facilities is to extend the National Addressing System and to improve the property ownership system. This will both improve postal services, as originally proposed, and provide better information for important revenue and financial analyses. Land use mapping and environmental impact assessments are important second priorities for GIS applications. Other features already developed for interim studies and analyses include mapping of significant coastal and wildlife features and environmentally sensitive areas from the Anegada Development Plan and mapping of important submarine habitats adjacent to Virgin Gorda.

The BVI have concentrated on developing GIS applications to address strategically important issues in the territory. Marine and coastal resources are vitally important to the BVI economy. They embody historical and cultural values, as well as maintain a high-quality environment to support charter yacht-based tourism, which is integral to the BVI economy. The Conservation and Fisheries Department is working with the Town and Country Planning Department to convert the country's coastal atlas to digital form (see section on Principal Users), as has been done on a demonstration basis for the Anegada and Virgin Gorda mapping.

U.S. Virgin Islands

In the USVI, environmental decision-making generally follows an adversarial, rather than a problem-solving format. A combination of historical and cultural factors have created the general assumption that the development process creates winners and losers. In this environment, information becomes an important tactical weapon, making it difficult to gather support for activities or programs that aim to make information more widely accessible. Technology is more acceptable, and more likely to receive leadership support, if it is justified on technical, less "political" terms.

The USGS team made a presentation on the Virgin Islands GIS in October 1992, after just completing the digital coverages for Phase I of the GIS project. The presentation emphasized that the GIS is intended to provide decision-makers with easily accessible spatial information (6).

Originally, the GIS was expected to benefit primarily the territory's three coastal zone commissions in their assessment of environmental effects of major development proposals. The ground-water protection program of the Division of Environmental Protection of the DPNR is using GIS analyses produced by the Water Resources Division of the USGS, employing the USVI GIS coverages with added data (e.g., wells) that the Water Resources Division is digitizing.

GIS Support Factors

According to the GIS support contractor for the BVI, a successful GIS requires three key support elements:

- GIS policy leadership
- GIS technical leadership
- Competent outside expert assistance

The following summarizes the comparative experience of the two programs for these three key implementation support factors:

Support Factors	BVI	USVI
GIS policy	Town and Country Planning Department director led project from chief minister's office	No GIS manager in government
GIS technician	BVI technician trained in the United States	No GIS specialist in government (draftsman at WAPA)
Outside support	UNDP and IDAS	USGS technical support and U.S. Department of the Interior financial support

Principal Users: Planned and Actual

British Virgin Islands

The BVI Department of Finance is the first user of data products from the GIS, based on the initial funding for the addressing system. This system is based on detailed parcel maps of the BVI so that the effective base map resolution of the BVI system is 1:2,500. This is considerably finer than the 1:24,000 scale of the USVI maps.

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The Town and Country Planning Department, however, is working to recruit other users to the system, including:

- Public Works.
- Water and Sewerage Department for a systemwide map (which may eventually spin off as a separate system, given this department's long-term interest in engineering-quality facilities management information).
- Conservation and Fisheries.
- British Overseas Development Administration, which funded a coastal atlas for the BVI (7). At the suggestion of the Town and Country Planning Department, this mapping was developed in ARC/INFO. A proposal has been made to convert the coastal atlas to digital form for natural resource management applications, with a demonstration already developed showing the distribution of sensitive marine communities around Virgin Gorda.

In addition to these uses, the GIS group is starting to experiment with the use of remote sensing products in GIS production, which would encourage the use of GIS for natural resource change detection.

The Town and Country Planning Department's GIS specialist, Mikey Farara, is being reassigned to provide networking support (including GIS distribution over the network) for several government agencies. Meanwhile, the GIS operation is adding a cartographer to assist in tailoring GIS products to users' needs.

As enthusiasm for the GIS has blossomed in the BVI, managing for realistic expectations and stressing the investment costs that participating agencies can expect have become problems for the Town and Country Planning Department.

U.S. Virgin Islands

Complex evaluation issues face the USVI's three coastal zone commissions (one on each island). Therefore, land use planning in general and coastal zone permitting specifically were assumed to be important first users of the GIS. The DNPR, however, had no process to prepare the Division of Comprehensive and Coastal Zone Planning to implement this system. In addition, the scale of permitting decisions may be too fine for the GIS base map. (See discussion of scale below.)

WAPA is not using the GIS data. One senior manager characterized their experience with the GIS project as "paying a lot of money for a diskette of data that we keep locked in the safe."

The VINP (part of the U.S. National Park Service) and Biosphere Reserve have purchased a PC-based GIS system and employed an analyst to implement it for the park and adjacent areas on St. John and the surrounding seas. With this system, they plan to enter the USGS-developed data into the database. In addition, the Virgin Islands Resource Management Cooperative (a collaboration of research and resource management organizations) makes the VINP GIS data and analytical capabilities available to members, including government members.

At this time, the only major Virgin Islands government user of the GIS is the ground-water protection group of the DPNR's Division of Environmental Protection. Because they lack equipment or software to manipulate the GIS data already available, they use the Water Resources Division of USGS as a GIS contractor. This arrangement has two problems:

- *High costs:* Although the USGS "owns" the existing digital data, and processing private contracts would be complex, DPNR believes it could get similar services at cheaper prices from other vendors.
- *Inappropriate scale:* Environmental management processes in the Virgin Islands (and in most other small island states) require knowledge of property ownership, implying maximum map scales of 1:5,000 to 1:10,000. The USGS quad sheet scale of 1:24,000 is too coarse for many management purposes. Costs of remapping areas of concern at the higher resolution are high, and the problems of maintaining multiple map resolutions and sources are not trivial.

What GIS Can Do

Joseph Berry has proposed seven basic categories of "What GIS Can Do for You" (8). These applications can be related to the GIS products and proposals for the BVI and USVI, with special attention given to natural resource and environmental issues. Table 3 shows what coverages that have been or are being developed for the two systems can do.

Table 3 illustrates two contrasting issues separating the two jurisdictions. The USVI have the data available to perform a number of relatively complex analytical processes, especially in St. John. They have no capability to actually execute any such studies, however. The BVI, on the other hand, have proposed and often developed pilot or demonstration applications for several GIS uses but still need to develop the data resources to support these on a territorywide basis.

Lessons Learned

The comparative experience of these two very distinct GIS programs reinforces three basic lessons of information system design and implementation:

- *Plan, don't assume:* The prolonged, sometimes repetitious, planning process that evolved in the BVI

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Table 3. Coverage Capabilities

Questions:	Analytical Function	USVI Application	USVI Status	BVI Application	BVI Status
Can you map it?	Mapping	USGS and WAPA-based coverages	Done	Land use cadastral	Done and proposed
Where is what?	Natural resource management	DLG hydrography	Done	Coastal atlas Sensitive areas	Proposed and partial
		DLG hypsography	Done	Significant features Population data	Major islands done
		Well inventory	Done	Land use	
		STJ national park coverages	Done	Sensitive areas	
Where has it changed?	Temporal	DLG and WAPA	Done	Land use updates population	Proposed
		Boundaries and roads 1982 to 1989			Proposed
What relationships exist?	Spatial			Land use Population data Coastal atlas	Partial Proposed Partial
Where is it best?	Suitability	STJ national park coverages (limited application)	Done	Land use Coastal atlas Sensitive areas Significant features	Partial
What affects what?	System	STJ national park coverages (limited application)	Done	Land use Coastal atlas Sensitive areas Significant features Population data	Partial and proposed
What if...?	Simulation	None discussed		Speculation, but no plans to implement yet	

involved multiple consultants providing often conflicting advice. This process served to educate policy-makers and managers in a much broader range of possibilities and avoidable problems than were available to the USVI. A corollary to the need for careful planning is the need to avoid making decisions or commitments to specific systems before such decisions are absolutely necessary. Especially in systems involving high technology, premature decisions often mean early obsolescence.

- *Implement in phases with early demonstration products:* Some issues, such as cadastral mapping and scale, are so subtle to inexperienced users that they need practice in real-life situations. If the USGS had spotted the scale problems at an early stage in the data conversion process, the USGS may have been able to provide a better solution. Some USVI critics claim the "1:24,000—one size fits all" attitude characterizes the federal approach.

- *Identify critical success factors for each situation:* In some environments (e.g., USVI), GIS is most attractive for its ability to provide enhanced powers of analysis. In others, such as the BVI, it is seen as a data integration tool and as a way to better inform political leadership and the public. To ensure success, major GIS implementations also need to meet the three major support requirements:
  - A political/senior management "chief"
  - A technical "chief"
  - Competent outside technical assistance

Finally, implementers should recognize that they have a stake in open information sharing. They should seek ways to redefine the decision-making process as a nonzero sum game: more information should benefit all parties. Of course, such changed attitudes require fundamental value shifts that take a long time to achieve and may have high short-term costs.

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R0021937

Application of GIS for Environmental Impact Analysis in a Traffic Relief Study

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Abstract

This paper presents an application of a geographic information system (GIS) in a traffic relief study. Traffic congestion has severely affected the environmental quality and the quality of life for residents in the study area. A team of planners, environmental specialists, historians, landscape architects, traffic engineers, and GIS professionals organized to solve the problem. The team has evaluated the environmental and socioeconomic impacts of highway alignments from the very first step through every major decision for the duration of the project.

The GIS professionals have played a crucial role in maintaining constant and active interactions among members of the project team, federal and state agencies, and the public. GIS has helped to develop a natural and cultural resource inventory, identify contamination sources, assess environmental constraints, and evaluate proposed highway alignment alternatives. GIS provides an ideal atmosphere for professionals to analyze data, apply models, and make the best decisions. The high-quality map products that GIS creates enhance the quality of public presentations and reports. The authors feel that, as this project has progressed, more people have realized the benefit of using GIS.

Introduction

A traffic relief study, as one type of transportation project, aims to resolve traffic congestion problems through a combined strategy of upgrading existing infrastructure, building new infrastructure, and controlling traffic demand using congestion management strategies (CMS). This type of study proceeds through at least the following steps:

- Problem identification
- Data collection
- Preliminary design
- Environmental impact analysis
- Final design
- Construction

The process heavily involves federal, state, and local government agencies, as well as the public. The goal of the project is to develop an environmentally sound solution to the traffic congestion, which also happens to promote economic development and improve quality of life for people in the local area and the region. Environmental, social, and economic issues must be equally addressed from the very first step through the final design. Federal and state regulations generally require an environmental impact statement (EIS) when constructing new infrastructure or upgrading existing road systems. Preparing an EIS is a requirement for such a project and demands a significant commitment of time, money, staff, and technical resources.

A geographic information system (GIS) has the ability to process spatially referenced data for particular purposes. Along with the development of computer hardware and software, GIS has progressed from pure geoprocessing, to management of geographic information, to decision support (1). This paper presents the application of GIS in an ongoing traffic relief study in Marshalls Creek, Pennsylvania. The GIS function in this study has had various purposes:

- Inventory data compilation
- Spatial data analysis
- Map production
- Traffic modeling support
- Public presentations

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The study shows that GIS can play an important and innovative role in transportation studies.

Project Description

The study area is in the Pocono region, located in northeastern Pennsylvania (see Figure 1). The Pocono Mountains and Delaware Water Gap National Recreation Area possess a wealth of natural and cultural resources. The area is famous for providing year-round vacation activities. Attractions include fishing, canoeing, and whitewater rafting in the summer and downhill and cross-country skiing, snowmobiling, snowboarding, and ice fishing in the winter. The area includes quiet woodland trails past a rushing waterfall and scenic settings for camping. In the fall, the area is ablaze with the brilliant colors of foliage. Various scenic sights, recreational sites, and national historic sites make the area ideal for attracting people to come for a day, a weekend, or a longer vacation.

Although tourism brings people to the Pocono area and promotes economic growth, it also brings a traffic congestion problem to the community. In addition, the influx of new home owners from New Jersey and New York adds the problem of commuter traffic to the area. The most troublesome section is in the vicinity of Marshalls Creek where U.S. Route 209 intersects with Pennsylvania (PA) Route 402. Two intersections are only about 500 feet apart. The traffic tieups can extend up to 3.5 miles on northbound Route 209, all the way back to Interstate Highway I-80. Emergency response times on U.S. Route 209 can be up to 20 minutes during peak traffic. The heavy traffic volume results in traffic accidents exceeding state averages on secondary roads as motorists seek alternative routes to avoid congestion. Through traffic traveling to and from New England using U.S. Route 209 as a connector between I-80 and I-84 makes the problem even worse. The year-round outdoor activities perpetuate the constant traffic problems that have severely affected the quality of life for residents in and around the Marshalls Creek area.

In response to the problems, the Pennsylvania Department of Transportation (PennDOT) selected a project team in February 1993 to conduct a traffic relief study in the Marshalls Creek area. The project team consists of individuals from seven firms and represents a wealth of experience in the variety of disciplines necessary to successfully complete this project. The team members include land use and traffic planners, biologists, historians, traffic and environmental engineers, surveyors, and GIS and global positioning system (GPS) professionals.

In addition to PennDOT, the funding agency, several federal and state regulatory agencies periodically review the development of the EIS to ensure that it meets regulations. These agencies are the Federal Highway Administration (FHWA), the U.S. Environmental Protection Agency (EPA), the U.S. Army Corps of Engineers, the Pennsylvania Department of Environmental Resources (DER), the Pennsylvania Historic Museum Commission, and the Pennsylvania Fish and Game Commission. Local planning commissions and citizen representatives also actively participate in advisory capacities. A series of agency coordination meetings, public meetings, and public information newsletters coordinates the activities of all participants over the course of the project.

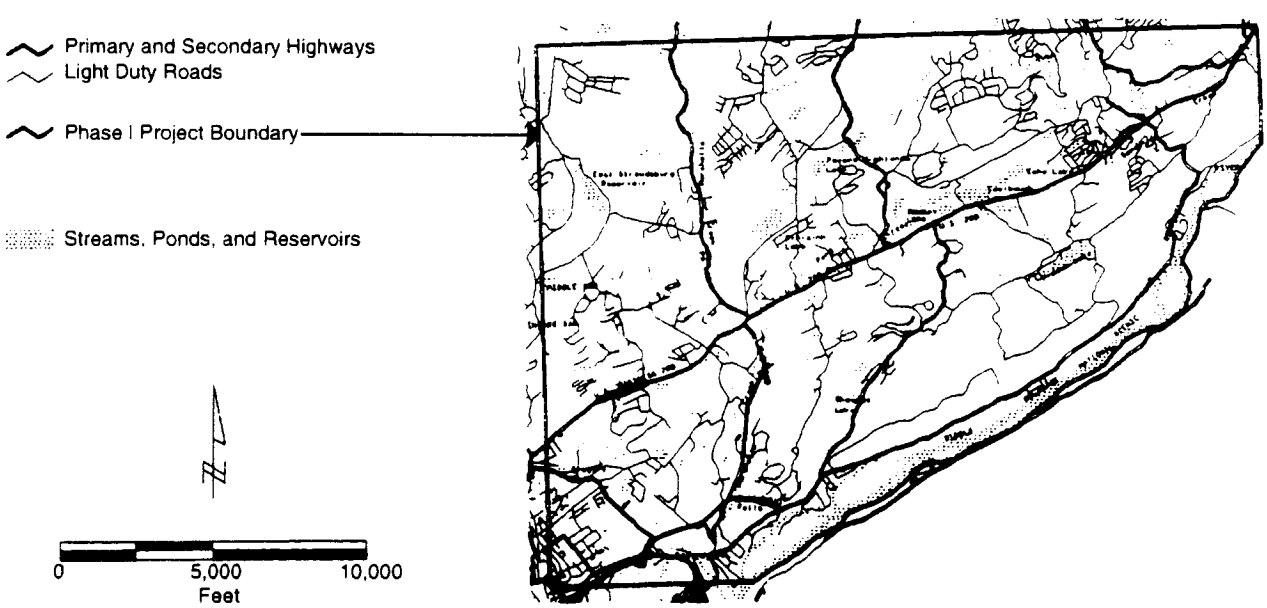

Figure 1. Phase I project area.

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After collecting traffic data and performing traffic demand modeling, the team realized that adopting strategies to control traffic demand and upgrading existing roads through widening and intersection improvements would not suffice to meet demand projections for the design year 2015. Consequently, the team has determined a new road is needed to alleviate congestion in Marshalls Creek.

The study aims to identify alternatives to relieve traffic congestion along U.S. Route 209, PA Route 402, and Creek Road, and eliminate backups onto I-80 from U.S. Route 209. The alternatives should also improve air quality by reducing fuel consumption and vehicle emissions and facilitate travel through Marshalls Creek for local and through traffic. The improvements must comply with federal and state regulations. The study team must consider county and local government goals and objectives so that the traffic capacity improvements will be compatibie with planned local development.

The project is being conducted in two phases. Phase I, which is complete, was an investigation broad in scope. It used inventory of secondary data to describe the environmental characteristics of the area. A traffic demand model identified the area for detailed study after a preliminary analysis of a wide range of baseline data. In Phase II, the team analyzes both primary and secondary data and delineates alternative alignments or transportation upgrading options that meet the need and minimize impacts. Analysis of environmental and engineering factors assists both in the determination of the most practical alternative and in preparation of the final EIS.

The nature of the study requires the analysis of a variety of data at different scales by different professionals. Through field investigation, the project team also constantly updates and adds new data to the existing database. The new data may be attribute data about some geographic features or may be locational data. The project team has found GIS to be an appropriate tool to meet the challenge of better conducting the study.

The team has used GIS extensively in both Phase I and Phase II studies. The two phases vary in data requirements, scales, and purposes of spatial analysis. With the support of GIS, the team has been able to quickly assemble data at adequate scales and present data in formats that are familiar to different professionals. GIS's data manipulation power distinguishes the different requirements of the two phases and at the same time, clearly depicts the linkage between the two phases. The following sections describe GIS applications that have helped facilitate the study and coordinate project team members, public agencies, and citizens.

GIS Application

GIS contains powerful tools to process spatially referenced data. These processes and their results are meaningless, however, without a clearly defined objective. Many professionals point out the importance of focusing GIS on practical problems. Using GIS is not an end; it is a means to represent the real world in both spatial and temporal dimensions. The benefits of using GIS can be summarized in three aspects:

- GIS helps to portray characteristics of the earth and monitor changes of the environment in space and time (2).
- GIS helps us to more deeply understand the meaning of spatial information and how that information can more faithfully reflect the true nature of spatially distributed processes (3).
- GIS helps us to model alternatives of actions and processes operating in the environment (2), to anticipate possible results of planning decisions (4), and to make better decisions.

This project demonstrates the advantages of applying GIS to solve practical problems from the above three aspects. An EIS requires extensive data about natural resources, land uses, infrastructure, and distribution of many interrelated socioeconomic factors. The accuracy and availability of required data depend on the scope of a study and the size of the study area. Our study shows that GIS, with its data retrieval, analysis, and reporting abilities, significantly improves the analysis. GIS helps to collect data at various scales, store data, and present data in forms that allow the project team to carry out the study in an innovative way.

Phase I Study

Phase I of the traffic relief study was completed in 1993. The goal of Phase I was to acquire understanding of the general features of the area and to use a traffic demand model for delineating an area for detailed study. The study area is approximately 52 square miles. To provide data for the preliminary analysis and the traffic demand modeling, the team developed baseline data inventory with GIS. Data were primarily secondary data that came from several different sources in different formats. For example, the U.S. Census Bureau 1990 population data were in TIGER format, the U.S. Fish and Wildlife Service National Wildlife Inventory (NWI) files in digital line graph (DLG) format, the U.S. Soil Conservation Service Monroe County Soils in DLG format, and the U.S. EPA Monroe County Natural Areas in ARC/INFO format. The majority of data sources were at scales between 1:15,000 and 1:24,000. With GIS tools, the team integrated these baseline data into a common presentation scale and projection. This process ensured an effective and comprehensive spatial analysis in the study area.

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The team arranged and stored data in layers according to themes. Examples of the data layers included:

- Road center lines.
- Twenty-foot elevation contours.
- Utility lines.
- Water features.
- Subdivision boundaries.
- 1990 Census population by Census Tracts and Blocks.
- Flood plains.
- Geological formations.
- Public facilities, including schools, churches, and cemeteries.
- Political boundaries.
- Hazardous waste locations.
- Potential archaeological areas.

With these data layers, the team generated a series of 17 thematic maps to describe the features of the study areas. All maps were plotted on E-sized papers (48 inches x 36 inches) with the same map layout. The general reference map served as a base map for the other themes. It included several data layers to provide geographic references to the study area.

In addition to the base map, each individual theme map showed only one theme at a time, such as soils, subdivisions, and wetlands. Some theme maps showed derived data from the original data layers. For example, in developing a slope theme map, the team first built a three-dimensional surface from the 20-foot elevation contours, then calculated slope in degrees and aggregated areas based on a 10-degree interval. The slope theme map showed the result from the data processing. In addition, the project team created summary statistics tables to help team members gain knowledge about the study area. Table 1 is an example of the summary statistics for land use categories.

Table 1. Phase I Statistical Summaries for Land Use Categories

Land Use	Acres
Urban	10,628
Agricultural	1,183
Rangeland	12
Woodland	20,957
Water	1,394
Wetland	1,428
Transitional	422

During preparation of the Phase I inventory data and summary statistics, traffic planners performed traffic demand modeling to determine new road connections that would provide a minimum acceptable level of service in the year 2015. The modeling result was loaded into the GIS and converted into the same format and projection as other inventory data. Figure 2 displays the boundary that the traffic demand model delineated and the actual Phase II boundary. The two boundaries were not the

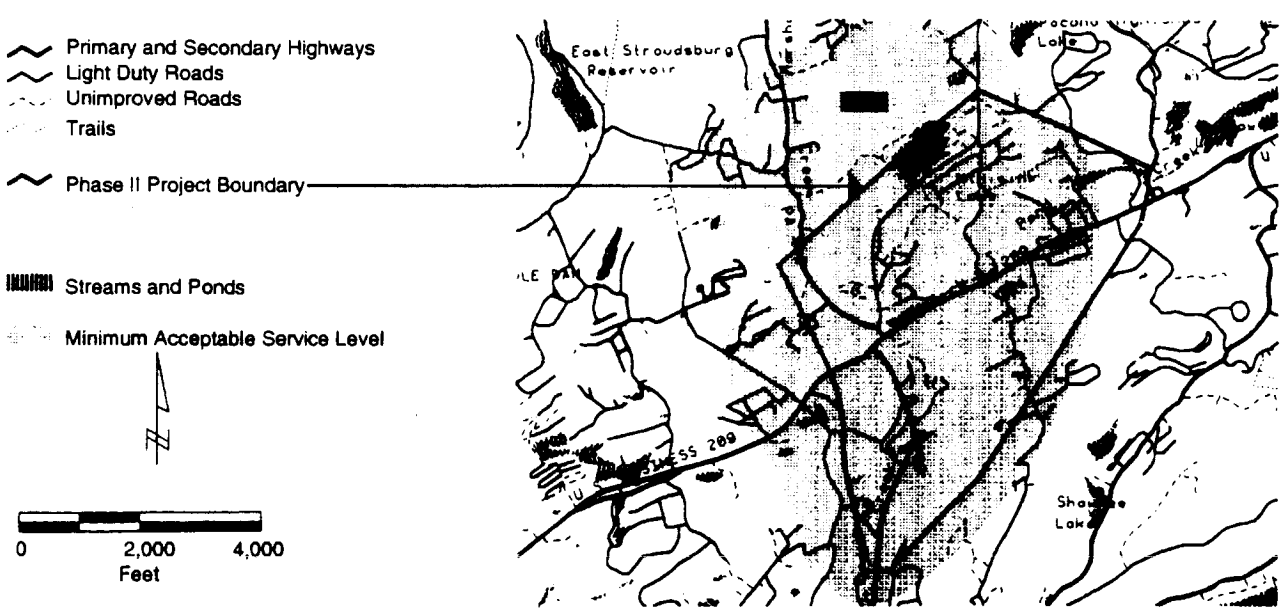

Figure 2. Phase II project area.

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same. The thick line enclosed a Phase II study area that was delineated based on the traffic demand modeling and the team's understanding of environmental and other factors in the project area.

Phase II Study

The Phase II study is still ongoing. The area for the Phase II study is much smaller than that for Phase I. It is about 3.2 miles by 2.5 miles, or approximately 4 square miles.

The objective of the Phase II study is to conduct a detailed analysis for delineating a full range of feasible highway alignment alternatives. The alternatives must meet the needs of relieving traffic congestion and minimizing its impact on environmental and cultural resources.

Because the accuracy of the Phase I data was not sufficient for the Phase II study, the project team has collected data using different approaches to develop a similar set of baseline data at a finer scale. The major data source has been the photogrammetry data provided by PennDOT at a 1:2,400 scale. The data include:

- Road cartways
- Five-foot elevation contours
- Utility lines
- Water features
- Buildings footprints
- Bridges

The team directly digitized tax parcel boundaries from Monroe County Tax Assessor's maps that range in scales from 1:1,200 to 1:4,800. After digitizing each map sheet separately, the team merged them together to create a continuous parcel layer. The data layer has been adjusted to fit with the PennDOT photogrammetry data although the two data sets do not seem to match exactly. In addition, digital orthophotographs at 5-foot pixel resolution were also obtained for the project.

From these baseline data, the team has constructed Phase II data layers in four different ways.

The first approach digitizes from compilations on project base maps. For example, the team creates a land use data layer from the digital orthophotographs and infrared photography. The GIS group first plots the digital orthophotographs on a set of 1:2,400-scale map sheets. Road cartway and water features are plotted on top of the orthophotographs. Then land use specialists delineate land use boundaries with fine color markers and code land uses on maps according to the Anderson land use classification. In the end, the GIS group digitizes the land use boundaries from the compilations to create a land use data layer. Similarly, the 100-year flood plain data layer is delineated from compilations on project base maps with 5-foot contours and digitized.

The second approach derives new data layers from existing data. Buildings and structures are plotted at a 1:2,400 scale. Both historians and environmental engineers use the plots in their field investigations. After historians identify historic-eligible buildings on the plots, the GIS group develops an attribute data file that links the historic inventory data to the building geometry. Similarly, field investigations identify buildings and structures associated with contamination sites. The system stores types of contamination as building attributes. By overlaying the building data layer with the tax parcel data layer, the team can identify properties on which historic buildings or contamination sites are located.

The third approach constructs data layers by referencing Phase I data. For example, Phase II subdivision boundaries are derived from the digitized tax parcel boundaries by referring to the Phase I subdivision boundaries. Phase I subdivisions were manually compiled at 1:24,000 using approximate location, which did not align very well with the more accurate tax parcel boundaries. Using a 1:7,200-scale plot that shows both Phase I subdivision boundaries and the Phase II tax parcel boundaries, planners can verify and indicate properties associated with each subdivision. These properties are dissolved to create new boundaries for subdivisions that precisely fit with tax parcels. The same approach is used to refine public parks and private recreation areas.

The fourth approach obtains spatial data with GPS. The GPS surveyors collect accurate locational data about key features, such as boundaries of wetlands, site locations for hazardous waste, and locations of archaeological field samples. GPS data also supplement existing data, such as delineating footprints of new buildings to update the PennDOT baseline data. The integration of GIS and GPS provides the project team with accurate and up-to-date data.

Phase II data layers have provided much richer information for a detailed study of environmental features. They are merged in many different combinations to show the spatial distributions of different factors from different perspectives. Table 2 lists some of the map themes created for the Phase II study. All maps are plotted at a 1:7,200 scale.

In addition to using GIS as a data library and map production tool, we use GIS to support decision-making in two ways. First, the creation of a composite data layer has revealed the impact of alternative alignments on several composite constraints. The composite data layer is an overlay of several inventory layers and shows the various factors coincident at any location, and the relative importance of these factors.

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Table 2. Selected Phase II Map Themes

Theme	Description
General reference	Road networks, hydrographic features, churches, municipal boundaries, and utilities
Parcels	Tax parcel boundaries
Community facilities	Public parks and private recreation, cemeteries, and public buildings
Flood plains	100-year flood plains
Land use	Current land uses
Subdivision	Approved subdivisions
Slope	Areas delineated with 5-degree slope intervals
Wetlands	Wetlands
Historic resources	Historic buildings and properties
Hazardous wastes	Hazardous waste sites and related buildings

These constraints have been identified by citizens, government agencies, and the project team. The major composite constraints include wetlands, historic properties, steep slopes (slope greater than 15 degrees), public parks and private recreation areas, 100-year flood plains, potential archaeological areas, prime agricultural land, subdivisions, and existing buildings and structures.

Two maps have been created from the composite constraints layer. One map shows the number of coincident constraint layers that occurs in any one location (see Figure 3). The other map shows the composite relative importance of coincident features. Both maps present a "sensitivity surface" view of the project area.

Secondly, the team uses GIS to perform interactive summary statistics for each alternative alignment. The project team analyzes the impact of the alignment alternatives on each individual constraint layer.

Two approaches define the impact areas for comparison. The first set impact areas are the areas enclosed by the footprint that traffic engineers delineated for each alternative. The second set impact areas are 300-foot buffers on both sides of the alignment delineated by the traffic engineers. The boundaries of the impact areas overlay on constraint data layers. Figure 4 displays wetlands crossed by alignment alternative ROW1B.

A set of summary statistics are calculated for each alignment. In the end, we compare the statistics for each alignment in a matrix (see Table 3). The matrix arranges constraints as rows and alignment alternatives as columns. The statistics include acres of selected features within each impact area, such as wetlands or high-quality watersheds, and total counts of features, such as historic-eligible buildings. The summary statistics also include listings of building names for businesses or public facilities within the impact areas.

The team has repeated the summary statistics several times as alignments shift. This procedure ensures that the final selected highway alignment minimizes environmental impacts, best meets project needs, and is the most cost-effective alignment to construct. The statistical matrix of impacts versus alignments is one of the critical evaluation criteria for comparing alignments and ultimately for selecting the final alignment.

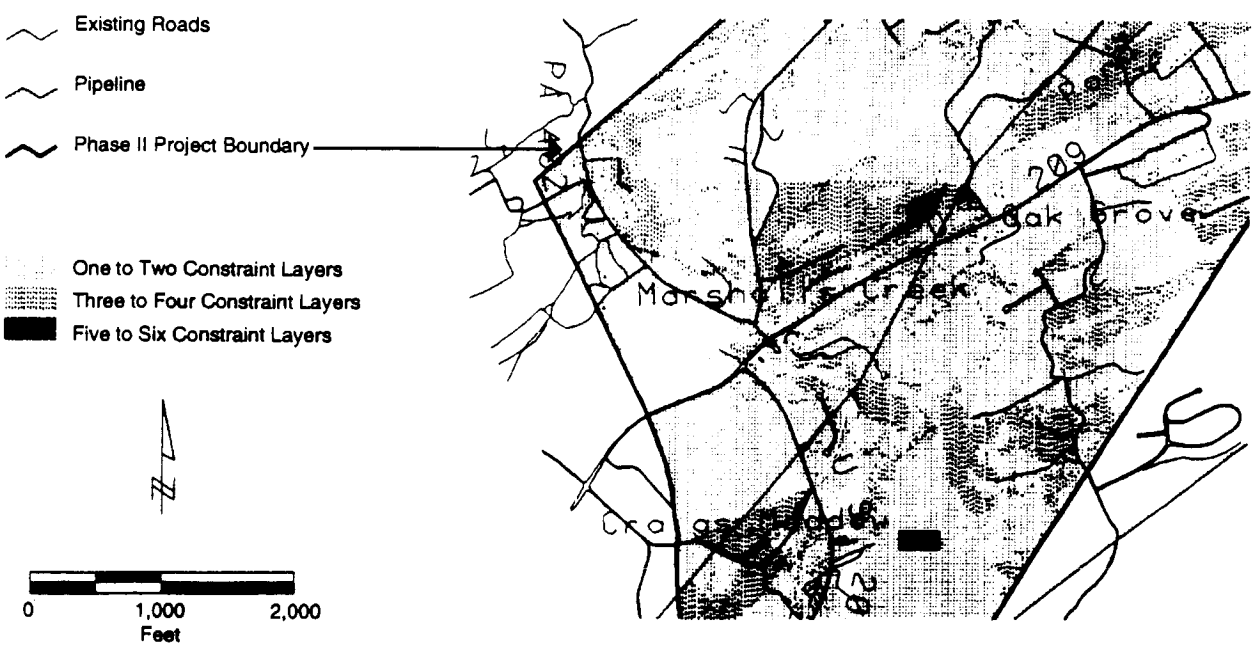

Figure 3. Composite constraints.

Table 3. Phase II Summary Statistics by Alternative

Constraints	Alignment Alternatives				
	ROW1A	ROW1B	ROW2A	ROW2B	ROW3
Wetlands					
Acres	2.76	3.27	3.25	1.8	13.64
Count	13	14	17	15	17
High-Quality Watersheds					
Acres	66.18	85.62	71.92	94.58	82.90
Hazardous Waste Parcels					
Acres	99.50	116.77	98.25	120.68	215.38
Count	8	5	11	10	6
Parcels With Historic-Eligible Buildings					
Acres	14.20	0.10	0.14	0.14	3.46
Count	3	1	2	2	2
Historic-Eligible Buildings					
Count	1	0	1	1	1

Benefit of Using GIS

In recent years, many federal, state, and local agencies have been actively acquiring and automating digital data (5). These databases provide various types of information at scales that are appropriate for a preliminary study covering a large area.

A more detailed study, which usually covers a smaller area, often requires more accurate data to describe the spatial distribution of relevant factors. GIS is flexible, allowing use of data at the scale and accuracy appropriate to the study purpose. The team has found that this feature helps improve the efficiency of the project without sacrificing the accuracy. This study has required two sets of scales ranging from 1:24,000 scale for Phase I, which required projectwide socioeconomic and environmental assessments, to 1:2,400 scale for Phase II, which requires detailed analysis for design of alternative alignments.

GIS has served as a digital database manager to assemble environmental, traffic, geographic, socioeconomic, and other data into a centralized project database. Data analyzed in this study originate from a variety of sources, such as PennDOT, U.S. Geological Survey (USGS), U.S. Census Bureau, Monroe County, and field survey. They are in different formats, including digital data in ARC/INFO, INTERGRAPH, and AutoCAD formats, GPS data, digital images, paper maps, and tabular data. Many of the public agencies and private organizations involved in this project already have digital data that GIS could easily use for this specific project. This has helped to reduce the overall costs of data collection and conversion.

This study demonstrates that GIS can support the information needs of many disciplines within a common framework and provide powerful, new tools for spatial analysis. Aside from technological considerations, GIS development initiates a higher-order systematization of geographic thinking (3), which is crucial to the success of a transportation project. In this project, GIS helps to determine the total impacts of alternative alignments on identified constraints. The team accomplishes this by superimposing the alternative alignments on constraint

Figure 4. Wetlands crossed by an alternative alignment.

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data layers to determine the total amount of each constraint layer that each alignment encounters. This approach supports the analysis of multiple alignments across the constraint surfaces for a variety of alternative scenarios. The spatial analysis tools and statistical function embedded in GIS prove to be very useful in such study.

Digital data that the GIS stores are used to summarize environmental, social, and economic data in many different ways. These functions include summing total acreage, listing entities of special interest, and counting numbers to provide useful baseline statistics for various alignments. Within a day, GIS accomplished what would have required several months of staff labor; GIS summarized impacts of 11 alternative alignments on all constraint layers. In addition, GIS creates composite constraints from individual data layers. A composite constraint data layer is created through a series of overlays to illustrate geographic coincidence of inventory themes.

Conventionally, engineers in a project such as this first delineate alternatives for alignments from the engineering perspective; they often consider factors such as steep slopes and costs. Then, other professionals, such as environmental specialists, historians, and planners, evaluate the alternatives from each point of view. GIS makes possible an early integration of environmental and engineering activities, ongoing communication with funding agencies and the public, and continual integration of a multidisciplinary team.

GIS helps to maintain high-quality data for the project. It allows for error checking and quality control of multiple data layers that would not be possible with conventional mapping. The team always compares a new data layer with other data to check for conflicts. Making check plots allows for quick identification of errors and missing data.

For instance, in the process of assigning building-use attributes to existing buildings, the GIS team first plotted buildings on a map and created a table with building identifiers. The field team then used the unique identifiers shown on the plots when noting building names and building uses in data collection tables. After relating the data table with the building data layer, the team found some buildings that did not have building use data. Moreover, some buildings were assigned uses that were out of range or seemed out of place, such as a residential building surrounded by several commercial buildings. The team highlighted the data for those buildings and sent them to engineers for verification. Through this data cleaning process, the team was able to obtain complete building use information.

Data quality directly affects project quality. Without GIS, this type of study often involves using and comparing maps at different scales, which frequently introduces serious errors. For each phase of this study, project team members have used a fixed-base map scale for all compilations. During data entry and data transformation, the team has kept accurate registration between data layers, ensuring data of the same resolution. In addition, the team has used FREQUENCY, one of the tools that GIS provides, to look for data values that are out of range as well as missing data. GIS tools have also helped to derive new relationships for features. For example, dissolving parcels has helped to create subdivision outlines, or overlaying historic buildings with parcels has helped to find the parcels on which they are located.

Planners, environmental specialists, historians, and landscape architects on the project team are responsible for field data collection, verification, and if necessary, compilation of field data into the standard project database. Wherever possible, the team has used GPS to eliminate the task of manual compilation and to improve accuracy of locating data. A fundamental requirement in applying the technology appropriately is to understand its capabilities, requirements, and limitations. Because several members manage inventory attributes, they need to know how to maintain unique identifiers for features so they can link up to the geometry. Because AutoCAD data transfers occur routinely with engineers, it is necessary to structure how the AutoCAD drawing files can be organized, as well as how certain attributes can be transferred by line color, layer name, or line width. The GIS group coordinates closely with other team specialists to identify quick, accurate, and cost-effective methods of data collection, data analysis, and presentation. In conjunction with the progress of the project, specialists from different fields have become familiar with the concept, requirements, and use of GIS. They now feel comfortable discussing alternatives while looking at results displayed on a computer screen.

GIS has created high-quality map products for public presentations and reports. GIS has also been used in several agency coordination meetings to display data and alternative alignments. GIS has allowed for different data layers to be displayed on the screen with specific combinations of features at various scales. Public agencies and citizens have been impressed by the clear and friendly graphic response to their questions. They have expressed interest in using the technology in future projects.

Once the study is complete, digital data assembled in this project will be an excellent resource for future projects in the study area. For these reasons, GIS provides more cost-effective project support for gathering, managing, and using data than that provided by paper and mylar maps.

Summary

For this project, the importance of innovation based on a solid scientific foundation cannot be overstated. In the

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current economic and regulatory climate, sound GIS methods are emerging as the only convincing and cost-effective means for locating, designing, and gaining approval for major public and private infrastructure projects.

The technology offers new and exciting tools for transportation planning.

The methodology that this project team has used can be successfully applied to other projects that require environmental assessment. The team has found GIS to be an extremely useful tool as users continue to learn its capabilities and the multiple tools that it offers. The regulatory agencies have repeatedly made favorable comments on how GIS can offer interactive viewing in a show-and-tell environment. This project is one of the first EIS projects to use GIS in a PennDOT-funded project. PennDOT appears convinced that GIS is an important component for conducting EIS for highway studies. More EIS projects probably will demand GIS services as part of the project approach, in part, because many federal and state regulatory agencies are increasingly using GIS.

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☆ U.S. GOVERNMENT PRINTING OFFICE: 1997 549-001/60118

R0021946

United States
Environmental Protection Agency
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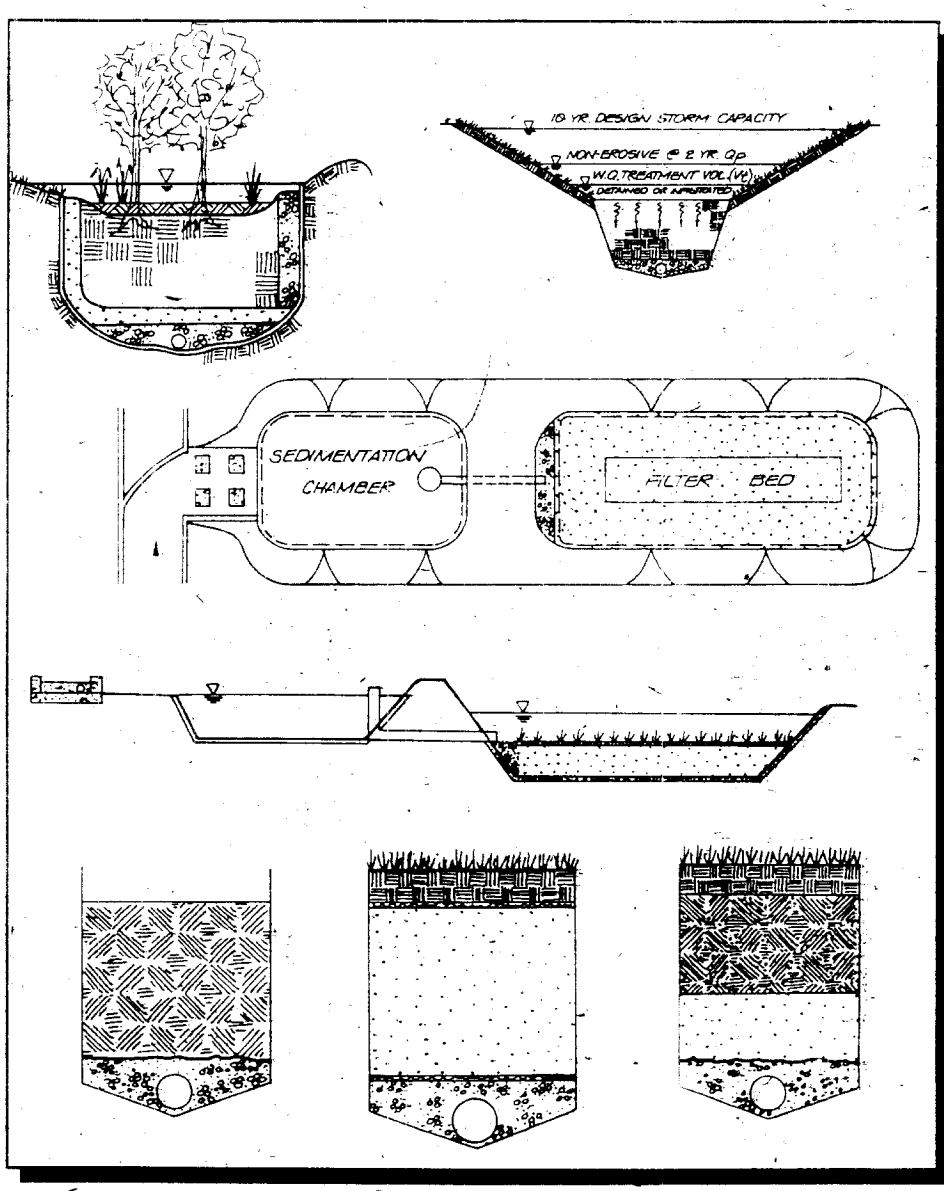
DESIGN OF STORMWATER FILTERING SYSTEMS



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PREPARED FOR:

CHESAPEAKE RESEARCH CONSORTIUM

CENTER FOR WATERSHED PROTECTION

R0021948

Abstract:

Title: Design of Stormwater Filtering Systems

Date: December, 1996

Authors: Richard A. Claytor and Thomas R. Schueler
Center for Watershed Protection

Publisher: Published under a cooperative agreement with the Chesapeake Research Consortium with supplemental funding by Region 5 of the Environmental Protection Agency by the Center for Watershed Protection. The Center for Watershed Protection is a non-profit organization dedicated to the protection, restoration, and stewardship of our nation's water resources and watersheds through sensitive management of the land. The Center promotes the advancement of innovative and effective land and water management techniques, and serves as a forum for planners, engineers, landscape architects, and municipal officials engaged in watershed protection.

Abstract: The project is oriented to create a unified design manual for stormwater filtering systems to remove pollutants from urban runoff generated at smaller sites within the Chesapeake Bay watershed. The primary audience for the manual are engineers, planners and landscape architects at the local or state level that need to comply with stormwater regulations in urban or suburban areas.

The manual presents detailed engineering guidance on eleven different filtering systems. The term stormwater filter refers to a diverse spectrum of stormwater treatment methods utilizing various media, such as sand, peat, grass, soil or compost to filter out pollutants entrained in urban stormwater. These filters are typically designed solely for pollutant removal, and serve small development sites. The three broad groups include: sand filters (surface, underground, perimeter, organic, and pocket designs), bioretention, and vegetated channels (grass channels, dry swales and wet swales, filter strips, and gravel wetlands).

The seven chapter design manual promotes a volume-based sizing criteria for all filtering systems utilizing principles of *small storm hydrology*, provides detailed guidance on the selection of appropriate filter types for various applications, reviews pollutant performance data and pathways for stormwater filtering systems, and provides detailed engineering design principles and guidance. The manual provides several design examples and contains over one-hundred tables and figures.

Price: $30.00

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Design
of
Stormwater Filtering Systems

Prepared by

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with supplemental funding by

U.S. Environmental Protection Agency, Region 5

The contents do not necessarily reflect the views and policies of the Chesapeake Research Consortium, Inc., or the Environmental Protection Agency, nor does the mention of trade names or commercial products constitute endorsement or recommendation for use.

December 1996

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CALIFORNIA REGIONAL WATER QUALITY CONTROL BOARD
LOS ANGELES REGION
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PREFACE

This project is oriented to create a unified design manual for stormwater filtering systems to remove pollutants from urban runoff generated at smaller sites within the Chesapeake Bay watershed. The primary audience for the manual are engineers, planners and landscape architects at the local or state level that need to comply with stormwater regulations in urban or suburban areas.

This manual continues the Center's efforts to produce urban stormwater practice design manuals targeted at specific categories of systems. Stormwater filtering is just one of these targeted areas. Existing and future manuals will cover areas such as wetland systems and pond systems.

Primary funding support for the preparation of this manual has been provided by a grant from The Chesapeake Resource Consortium with supplemental funding by Region 5 of the U.S. Environmental Protection Agency to complete Chapter 6, and the appendices.

ACKNOWLEDGMENTS

Preparation of this manual would not have been possible without the contributions from many experts in the stormwater management filtering field. The authors would like to extend our appreciation and acknowledge the following individuals for their contributions and constructive comments: Robert Pitt, Warren Bell, Richard Horner, Earl Shaver, Hung Van Truong, Larry Coffman, and John Galli. We would expressly like to thank Earl, Warren and several of their co-workers for taking time out of their busy schedules to show us several examples of stormwater filters and their adaption to real world problems. This provided a much welcomed "reality check" for some of our design parameters contained herein.

Special gratitude to Don Koch of Engineering Technologies Associate for his contributions on the bioretention water balance analysis, Tim Schueler and Rick Scaffidi of Environmental Quality Resources for their insightful comments and suggestions, Ricardo Gonzalez and Jeffery Everhart for their diligent work on the grass species selection guide, Keith Bowers for his contributions to the bioretention species selection guide, Dean Geiser and Donna deMars for their superb preparation of graphic figures and drawings, and Arlene Allegretto for her tireless efforts in making this manual a reality.

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INTRODUCTION

The manual presents detailed engineering guidance on ten different filtering systems. The term stormwater filter refers to a diverse spectrum of stormwater treatment methods which utilize an artificial media , such as sand, peat, grass, soil or compost to filter out pollutants entrained in urban stormwater. These filters are typically designed solely for pollutant removal (quantity bypassed), and serve small development sites (usually less than five acres). The three broad groups include: sand filters (surface, underground, perimeter, organic, and pocket designs), bioretention and vegetated channels (grass channels, dry swales wet swales, and filter strips).

The underlying concept of the manual is that a common and unified approach was needed to design each type of stormwater filter, so that this useful technology can gain wider engineering acceptance at the local level. Therefore, each stormwater filter incorporates four standard engineering features: a flow regulator, a pretreatment mechanism , filter media and bed specification, and overflow channels. In addition, the manual presents a single volumetric sizing requirement for each filter which is to capture and treat 90% of the runoff producing events that occur each year.

Many prior design approaches had been rate-based, and resulted in limited and unreliable pollutant removal rates. A third feature of the manual is that it utilizes new techniques for calculating runoff rates and volumes that reflect small storm hydrology from small, heterogeneous urban sites. Field research has indicated these methods are superior to traditional applications of the National Resource Conservation Service (NRCS) runoff forecasting models (such as TR-55 and TR-20). The manual also includes numerous step-by-step design examples that help an engineer apply the new design techniques. Lastly, the manual synthesizes recent research and field experience on the pollutant removal performance, longevity, cost, and maintenance burden of each type of stormwater filter, drawn from a national literature and phone survey. This information has been condensed in a series of tables that help designers and municipal officials select the most effective stormwater filter for their situation, and compare the performance of stormwater filters to that of other stormwater BMP options (e.g., ponds, wetlands, and infiltration systems).

Although stormwater filters can be applied to a diverse range of development conditions as a group, individual designs are limited to a more narrow range of site

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conditions . The most economical and feasible options are identified for five broad categories of development: ultra-urban, parking lots, roads, residential subdivisions, and backyard/rooftop drainage. Key feasibility factors that influence the selection of stormwater filters include space consumption, minimum head, maintenance burden, cost/acre and soil conditions.

During the study, over thirty published and unpublished studies on the pollutant removal performance of stormwater filtering systems were consulted (and are abstracted in Appendix A and cited in the References). Estimated removal rates for each of the stormwater filters are derived in Chapter 4, based on monitoring studies, infiltration rates, modeling and inference from similar technologies. Despite their many differences in design, stormwater filters have many similarities in performance.

The performance, feasibility, and environmental restrictions of stormwater filters are compared to three other groups of stormwater BMPs that are currently in widespread use by engineers in the Chesapeake Bay region-ponds, wetlands and infiltration systems.

In general, stormwater filters are the most feasible option for smaller development sites (less than 5 acres) but are not typically cost effective beyond that drainage area. Other BMPs, most notably ponds and wetlands, also have higher or more reliable removal rates for nutrients, bacteria and hydrocarbons. Ponds and wetlands, however, cannot usually be applied on small development sites and ultra urban conditions. Another key advantage of stormwater filters as a group is their lack of environmental drawbacks, such as stream warming, groundwater contamination, wetland impairment, and public safety. On the other hand, with one notable exception (bioretention), most stormwater filters confer few if any amenity values to the community (such as habitat, flood control, landscaping or increase in property value.

In summary, stormwater filters have their greatest applicability for small development sites, and can generally provide reliable rates of pollutant removal if design improvement are made and regular maintenance is performed. Stormwater filters appear to have particular utility in treating runoff from urban "hotspot" source areas such as commercial parking lots, vehicle service centers, and industrial sites, as well as problematic street and highway sites when other BMPs are not feasible.

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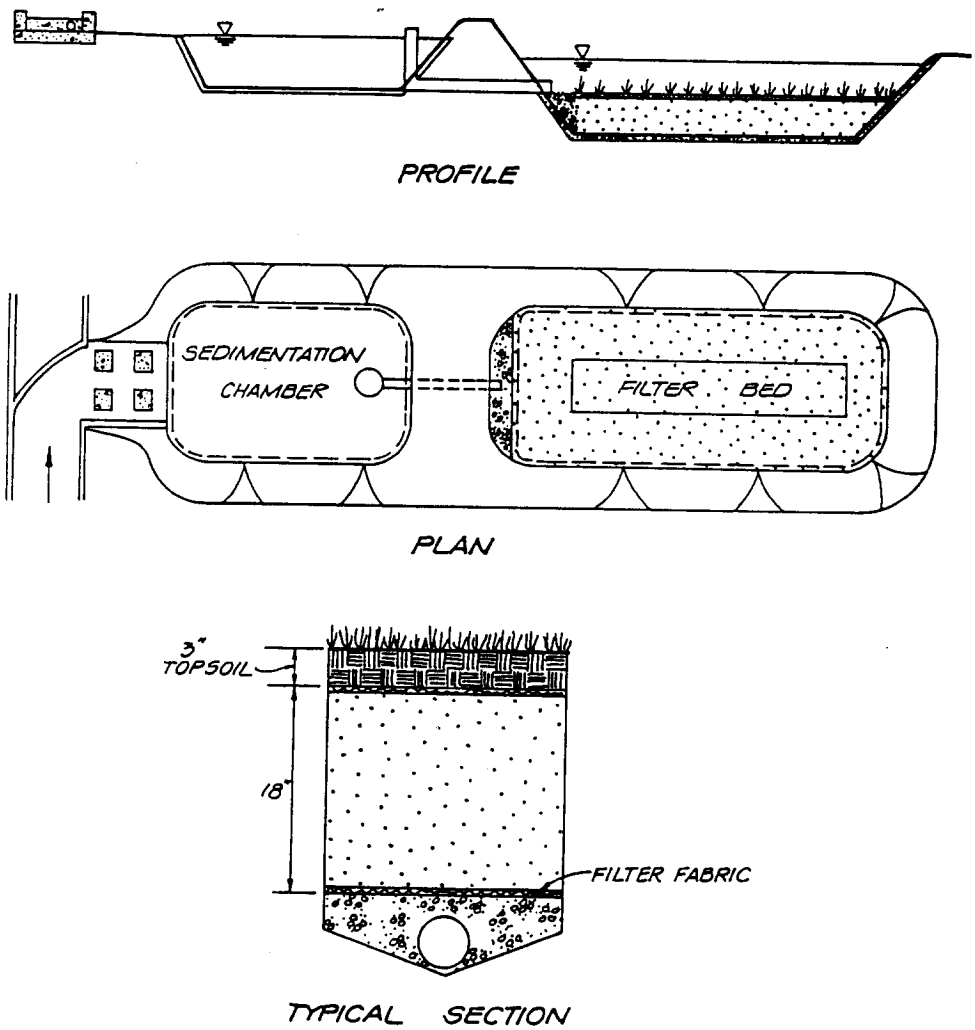

SURFACE SAND FILTER

Chapter 1

Introduction to Stormwater Filtering Systems

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CHAPTER 1
INTRODUCTION TO STORMWATER FILTERING SYSTEMS

1.1 *WHAT ARE STORMWATER FILTERING SYSTEMS?*

Stormwater filtering systems refer to a diverse group of techniques for treating the quality of stormwater runoff. The common thread is that each utilizes some kind of filtering media, such as sand, soil, gravel, peat or compost to filter out pollutants entrained in stormwater runoff. In addition, most filtering systems are typically applied to small drainage areas (five acres or less). Third, filtering systems are designed solely for pollutant removal. Flows greater than the water quality treatment volume are bypassed around the filter to a downstream stormwater management facility. Lastly, filtering systems incorporate four basic design components in every application.

1.2 *COMMON DESIGN COMPONENTS*

While stormwater filters are a diverse group of stormwater practices, they have several common design components. The four basic design components of a filtering system are: (a) inflow regulation that diverts a defined flow volume into the system; (b) a pretreatment technique to capture coarse sediments; (c) the filter bed surface and unique filter media, and (d) an outflow mechanism to return treated flows back to the conveyance system and/or safely handle storm events that exceed the capacity of the filter. Each of the design components are described in greater detail below:

1.2A INFLOW REGULATION

The inflow regulator is used to divert runoff from a pipe, open channel or impervious surface into the filtering system. The inflow regulator is designed to divert the desired water quality volume into the filter, and also allow large flow volumes to continue through the conveyance channel. With a few exceptions, most filtering systems are constructed *off-line* (i.e., runoff is diverted from the main conveyance system, treated, and then returned back to the conveyance system (Figure 1.1a). A few filtering systems are constructed *on-line,* such as the swale system depicted in Figure 1.1b. On-line filters are located within the conveyance system, and are exposed to the full range of flow events from the smallest storm up to and including the 100 year event.

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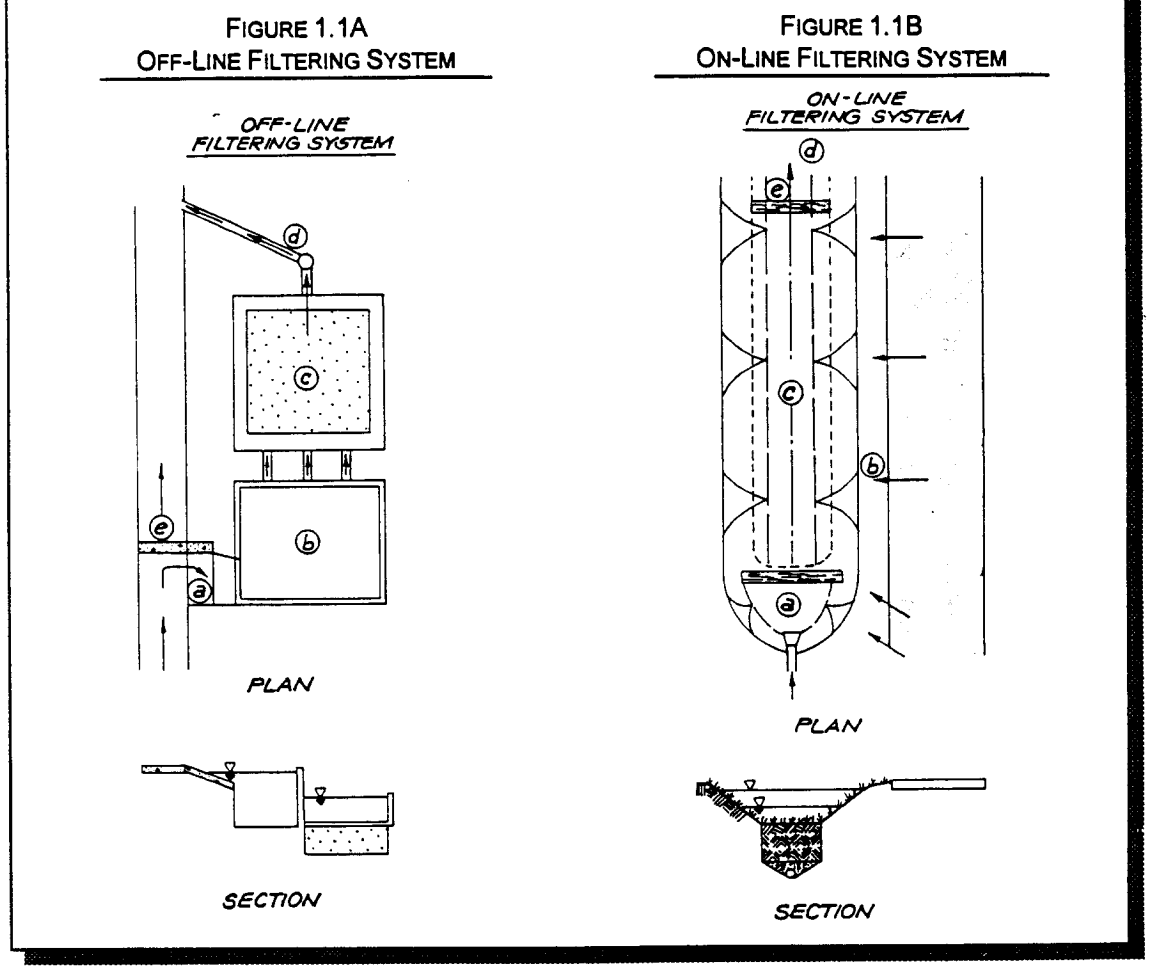
FIGURE 1.1: SCHEMATIC: ON-LINE VS. OFF-LINE DESIGN



1.2B PRETREATMENT

The second key component of any filtering system is pretreatment. Pretreatment is needed in every design to trap coarse sediments before they reach the filter bed.

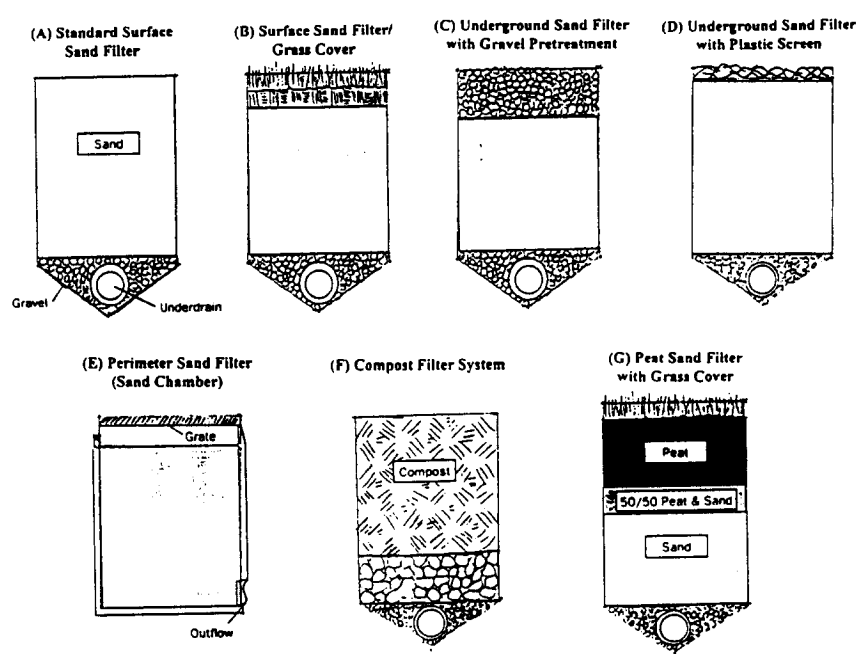
Without pretreatment, the filter will quickly clog, and lose its pollutant removal capability. Each filter design differs with respect to the type and volume of pretreatment afforded. The most common technique of pretreatment is a wet or dry settling chamber. Geotextile screens, pea gravel diaphragms and grass filter strips may also be used as a secondary form of protection. Sediments deposited in the pretreatment chamber must be periodically removed to maintain the system.

1.2c FILTER BED AND FILTER MEDIA

Each filtering system utilizes some kind of media such as sand, gravel, peat, grass, soil or compost to filter out pollutants entrained in urban stormwater, and some designs utilize more than one. The selection of the right media is important, as each has different hydraulic, pollutant removal and clogging characteristics.

The filter media is incorporated into the filter bed. The three key properties of the bed are its surface area, depth, and profile. The required *surface area* for a filter is usually based as a percentage of impervious area treated and the media itself, and may vary due to regional rainfall patterns and local criteria for water quality treatment volumes. The *depth* of most filtering systems ranges from 18 inches to four feet. A relatively shallow filter bed is used for hydraulic and cost reasons, and because most pollutants are trapped in the top few inches of the bed. Each design also utilizes a slightly different *profile* through the bed. An example of the variation in sand filter profiles is shown in Figure 1.2. As can be seen, each design has slightly different surface protection and layering through the bed.

FIGURE 1.2: CROSS-SECTION OF SAND FILTER DESIGN VARIATIONS



1.2D OUTFLOW MECHANISM

The final component of any stormwater filter design is the method(s) used to *collect or exfiltrate* the filtered runoff that leaves the filter bed and bypass the larger storm flows. The two primary methods for handling filtered runoff are to *collect* it in perforated pipes and return it back to the conveyance system, or to allow it to *exfiltrate* into the underlying soils where it may ultimately reach groundwater. Each method has its pros and cons. In the *collection* method, the bottom of the filter bed may be sealed with an impermeable liner which allows the filtered runoff to be captured in pipes and returned to the conveyance system. This is desirable if the contributing land use is considered a pollutant hotspot or if groundwater contamination is a concern. In the *exfiltration* method, the bottom of the filter bed is fully or partly permeable, and the filtered runoff continues downward through the soil and into groundwater. The uncollected runoff volume and pollutant mass drain into underlying soils and the water table. The advantage of exfiltration is that it provides groundwater recharge and takes advantage of the natural filtering capacity of soil to remove additional pollutants.

1.3 TYPES OF STORMWATER FILTERING SYSTEMS

This section describes the five broad groups of filtering systems that can be used for stormwater treatment. They include sand filters, open vegetated channels, bioretention areas, filter strips and submerged gravel filters. Within each group of filters are a number of important design variants that need to be considered.

1.3A SAND FILTERS

The City of Austin, Texas first pioneered the use of sand filters to treat urban stormwater runoff in the early 1980's. Since then the practice has rapidly evolved, with nearly a dozen variants of the basic sand filter design developed in response to different climatic, development and site conditions. For purposes of this manual, sand filter designs are grouped into five broad categories:

- Surface Sand Filter
- Underground Sand Filter
- Perimeter Sand Filter
- Organic Filter
- Pocket Sand Filter

SURFACE SAND FILTER

The earliest design was the surface sand filter, shown in Figure 1.3. A flow splitter is used to divert the first flush of runoff into an off-line sedimentation chamber. The chamber may be either wet or dry, and is used for pretreatment. Coarse sediments drop out as the runoff velocities are reduced. Runoff is then distributed into the second chamber, which consists of an 18 inch deep sand filter bed and temporary runoff storage above the bed. Pollutants are trapped or strained out at the surface of the filter bed. The filter bed surface may have a sand or grass cover. A series of perforated pipes located in a gravel bed collect the runoff passing through the filter bed, and return it into the stream or channel at a downstream point. If underlying soils are permeable, and groundwater contamination unlikely, the bottom of the filter bed may have no lining, and the filtered runoff may be allowed to exfiltrate.

UNDERGROUND SAND FILTER

The underground sand filter was adapted for sites where space is at a premium. In this design, the sand filter is placed in a three chamber underground vault accessible by manholes or grate openings. (Figure 1.4). Pioneered in the District of Columbia, the vault can be either on-line or off-line in the storm drain system. The first chamber is used for pretreatment and relies on a wet pool as well as temporary runoff storage. It is connected to the second sand filter chamber by an inverted elbow, which keeps the filter surface free from trash and oil. The filter bed is 18 inches in depth and may have a protective screen of gravel or permeable geotextile to limit clogging. During a storm, the water quality volume is temporarily stored in both the first and second chambers. Flows in excess of the filter's capacity are diverted through an overflow weir. Filtered runoff is always collected, using perforated underdrains that extend into the third "overflow" chamber.

PERIMETER SAND FILTER

The "Delaware" sand filter, developed by Shaver and Baldwin (1991), consists of two parallel trench-like chambers that are typically installed along the perimeter of a parking lot (Figure 1.5). Parking lot runoff enters the first chamber which has a shallow permanent pool of water. The first trench provides pretreatment before the runoff spills into the second trench, which consists of an 18 inch deep sand layer. During a storm event, runoff is temporarily ponded above the normal pool and sand layer, respectively. When both chambers fill up to capacity, excess parking lot runoff is routed to a bypass drop inlet. The remaining runoff is filtered through the sand, and collected by underdrains and delivered to a protected outflow point.

R0021968

ORGANIC FILTER

The organic filter functions the same as a surface sand filter design, with the exception that it uses compost or peat/sand as the filter media. The basic design of an organic filter is shown in Figure 1.6. A flow splitter diverts runoff into a pretreatment chamber, and then passes into a series of filter cells. Each filter bed contains an 18 inch layer of compost or peat, followed by a filter fabric, and six inches of perforated pipe and gravel. Runoff filters through the organic media and is then collected by a perforated pipe and directed toward the outlet. In most organic filters, the filter bed and subsoils are separated by impermeable polyliner to prevent movement into groundwater.

POCKET SAND FILTER

The pocket sand filter is a simplified and low cost design that may be used on smaller sites. Runoff is diverted within a manhole (Figure 1.7). A bypass pipe sends excess runoff along the storm drain system, and a flow diversion pipe routes the water quality volume into the system. Pretreatment is provided by a concrete flow spreader, a grass filter strip and a plunge pool. The filter bed is also a relatively simple affair. A shallow basin is excavated, and contains the sand filter layer. Most of the water quality volume is temporarily stored above the filter bed. The surface of the filter bed contains a soil layer and grass cover crop. In the event of clogging, the pocket sand filter has a pea gravel "window" to direct runoff into the sand, as well as a cleanout and observation well. In most cases, the filtered runoff is allowed to exfiltrate into the underlying soils, although underdrains may be needed if the soils are not suitably permeable.

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FIGURE 1.3: SURFACE SAND FILTER

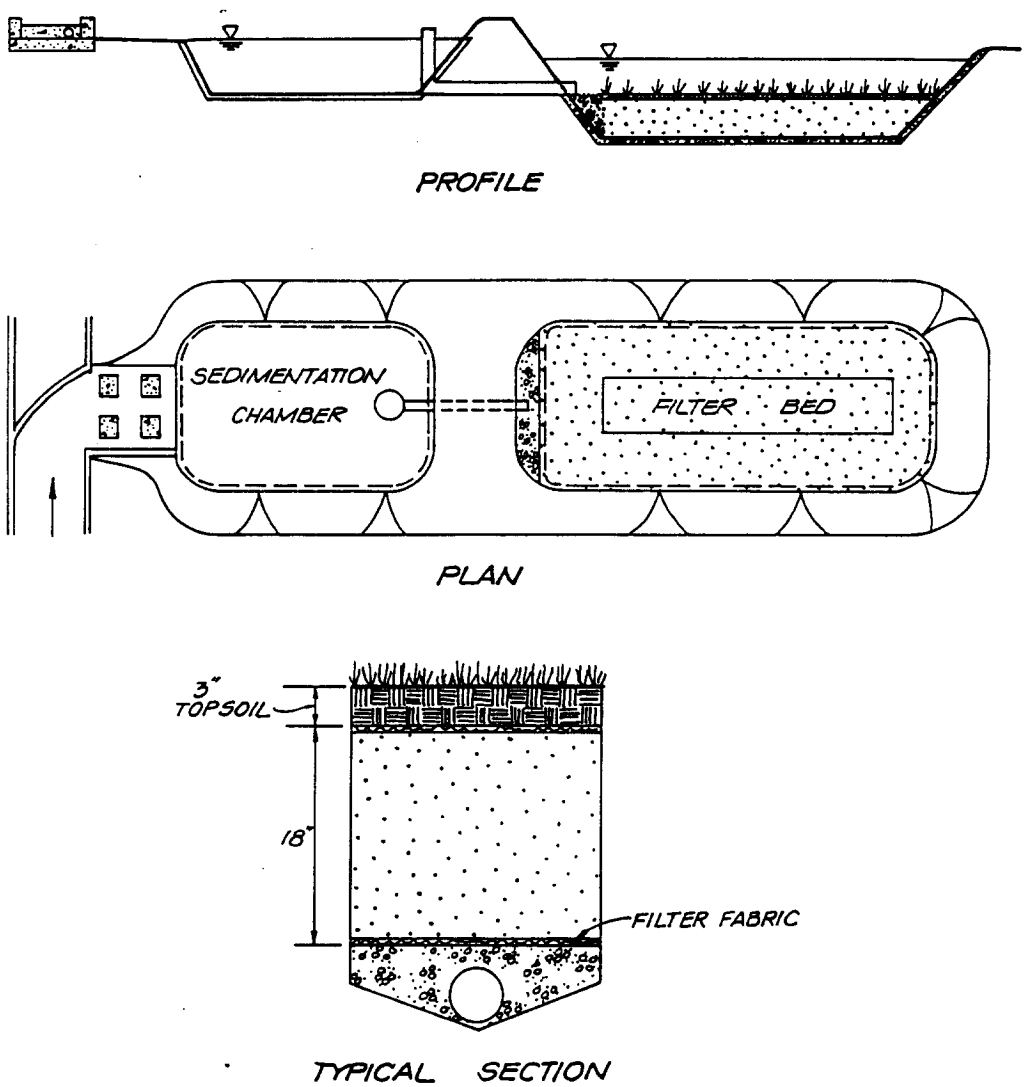


FIGURE 1.4: UNDERGROUND SAND FILTER

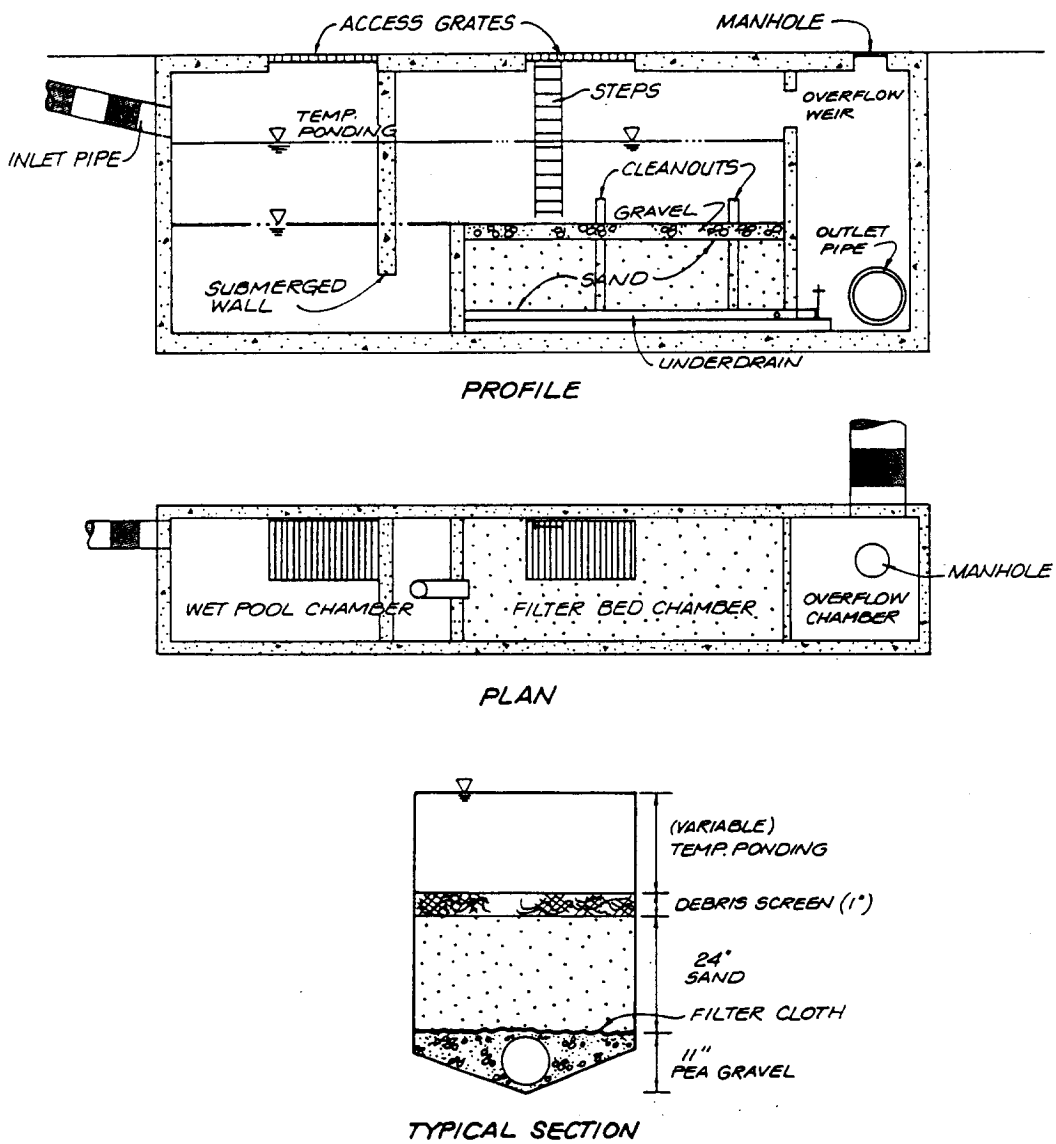


FIGURE 1.5: PERIMETER SAND FILTER

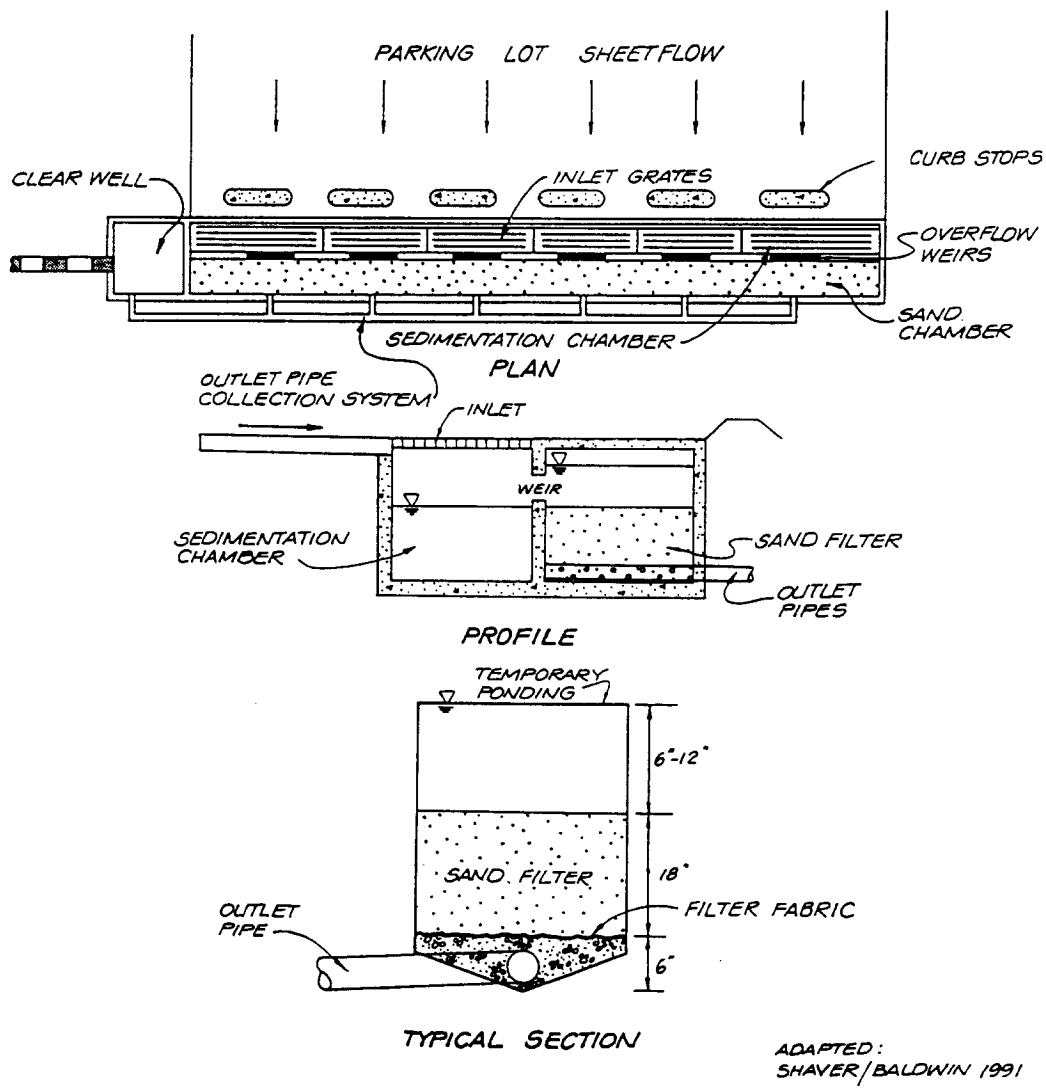


FIGURE 1.6: ORGANIC FILTER

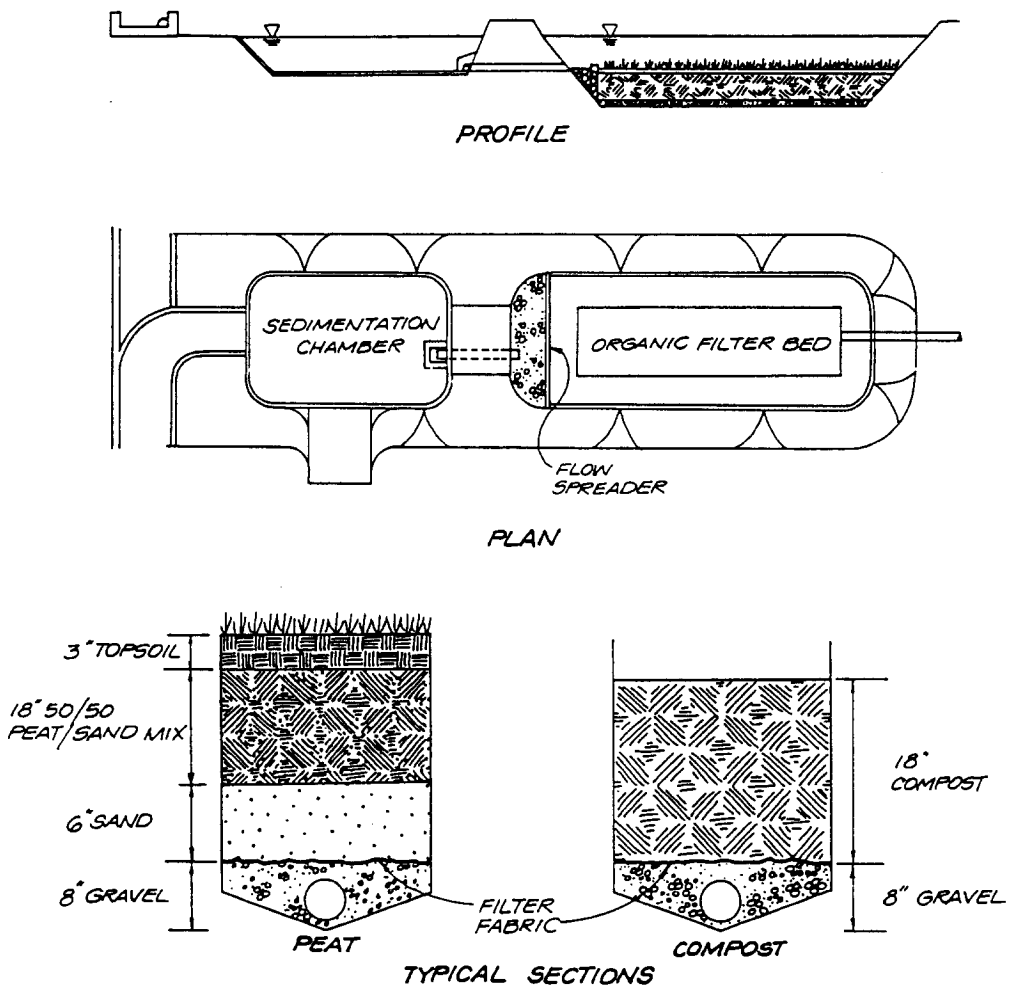
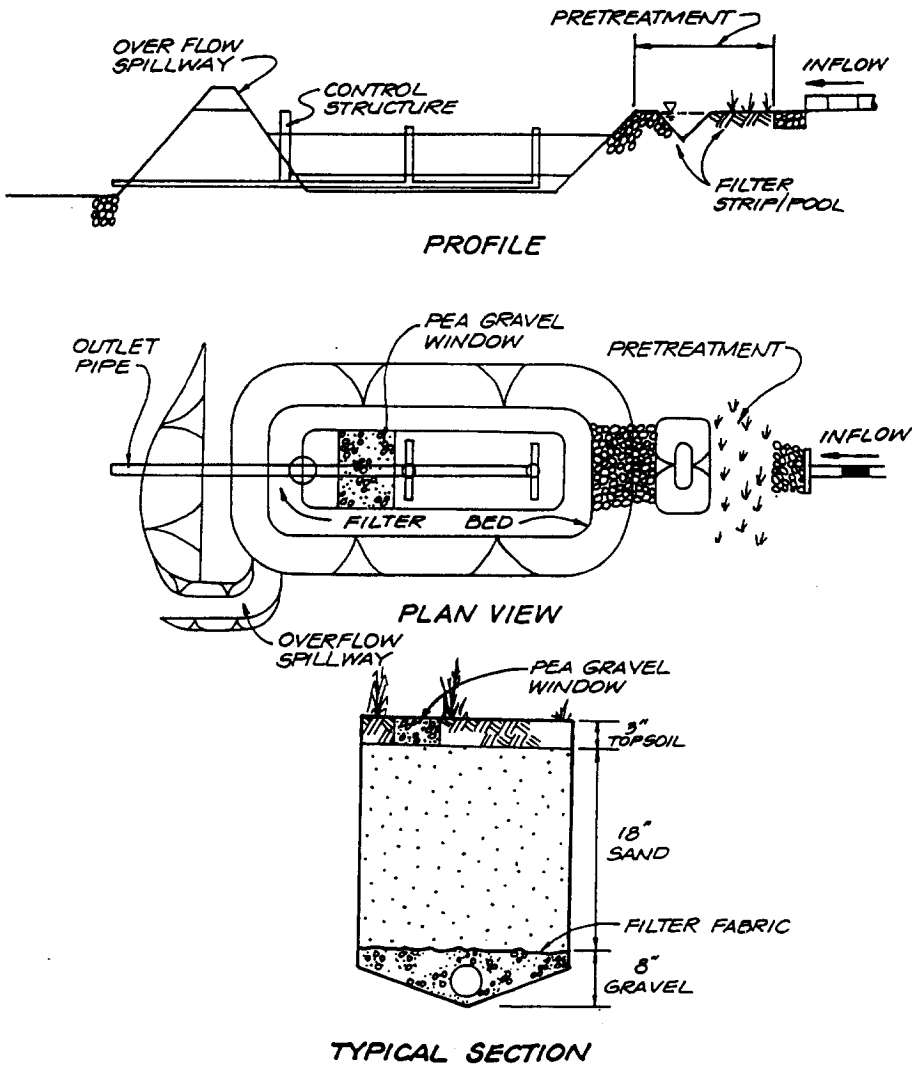


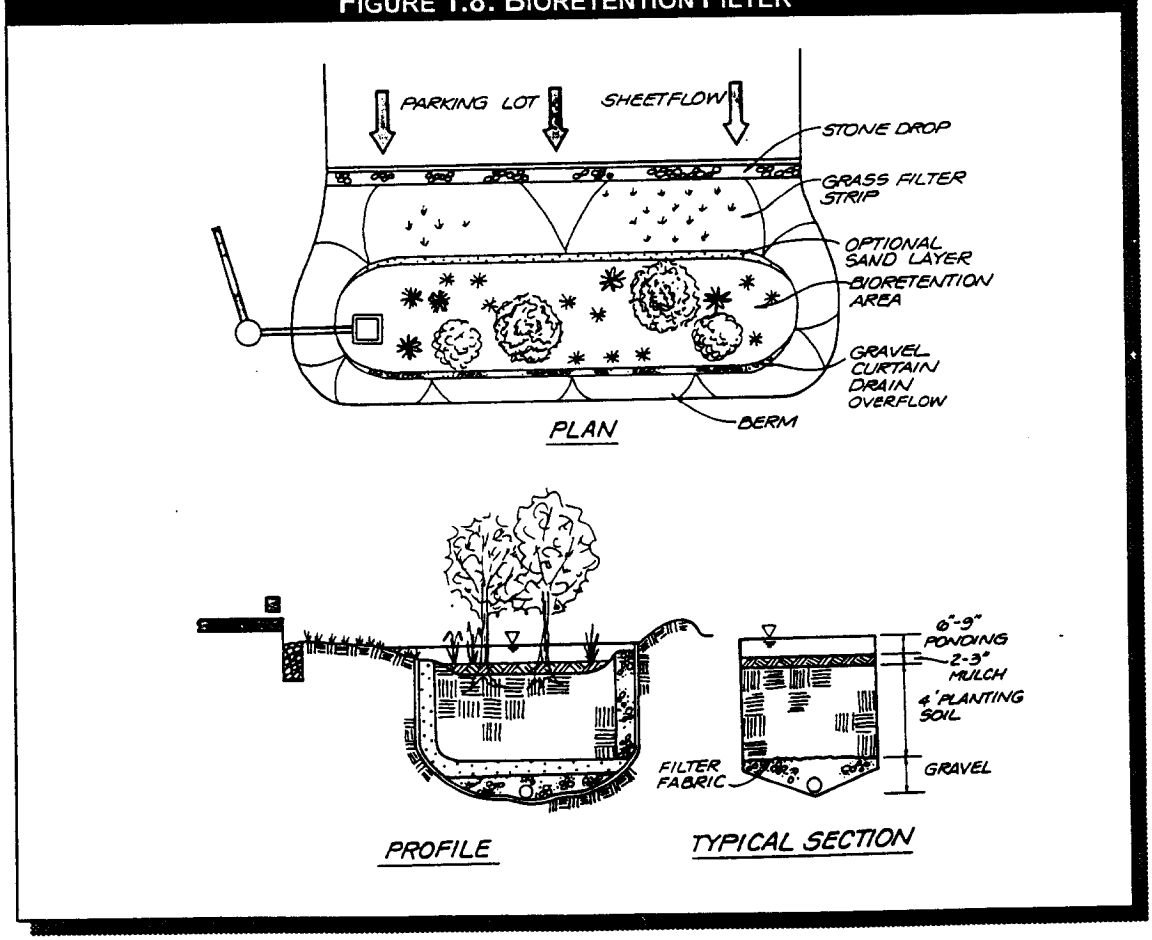
FIGURE 1.7: POCKET SAND FILTER



1.3B BIORETENTION

This filtering system utilizes parking lot islands and planting strips for on-site treatment of the water quality volume. Surface runoff is directed into shallow, landscaped depressions in the parking lot, known as bioretention areas. These depressions are modeled to incorporate many of the pollutant removal mechanisms that operate in forested ecosystems. Key elements include a grass filter, sand layer, loamy soils, mulch layer, shallow ponding of stormwater and plantings of native trees and shrubs (Figure 1.8). Pretreatment mechanisms include a stone drop at the edge of the parking lot that leads over a grass filter strip and a sand layer. During storms, the water quality volume is ponded up to nine inches above the mulch. Runoff in excess of the water quality volume rises to a higher elevation, but is then diverted into a standard drop inlet connected to the storm drain system. The remaining runoff filters through the mulch and prepared soil mix, which is about four feet in depth. Typically, the filtered runoff is collected in a perforated underdrain and returned to the storm drain system.

FIGURE 1.8: BIORETENTION FILTER

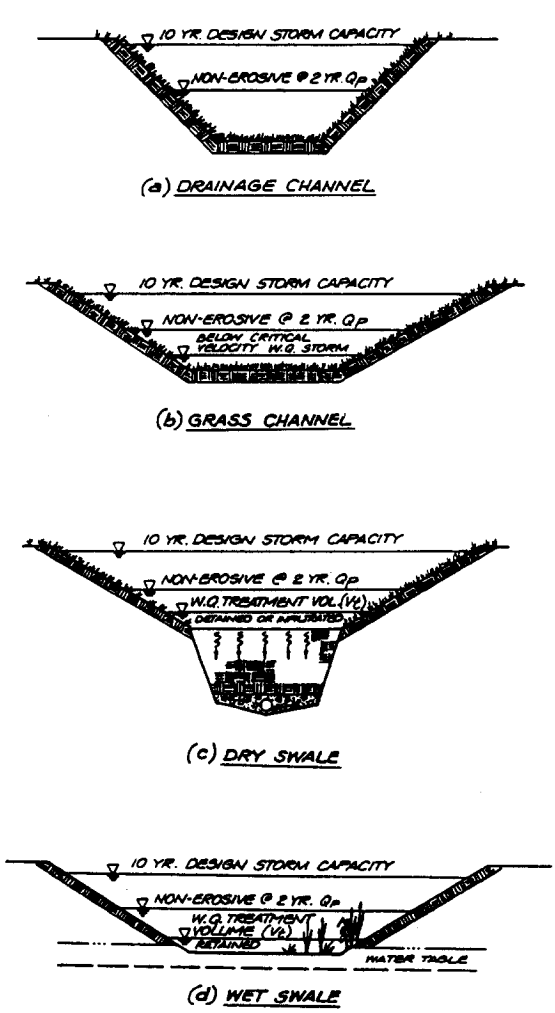


The benefits of bioretention include low land consumption, as the entire bioretention area can fit within the 5 to 10% of a parking lot that is typically devoted to landscaping. In addition, regular maintenance can be provided by commercial landscaping companies, and the "planting hole" provided by the bioretention area often increases the survival rates of landscaping.

1.3c OPEN VEGETATED CHANNELS

Stormwater engineers frequently use open channels or grass swales to convey stormwater runoff. In some cases, open channels can be redesigned to provide significant pollutant removal. It is therefore quite important to define what is meant by open channels, so as to better distinguish the potential differences in pollutant removal potential that various channel designs can have during small storms. In this sense, open channels can be classified into one of four possible categories, based on their hydrologic design. They are the drainage channel, grassed channel, dry swale and wet swale (Figure 1.9).

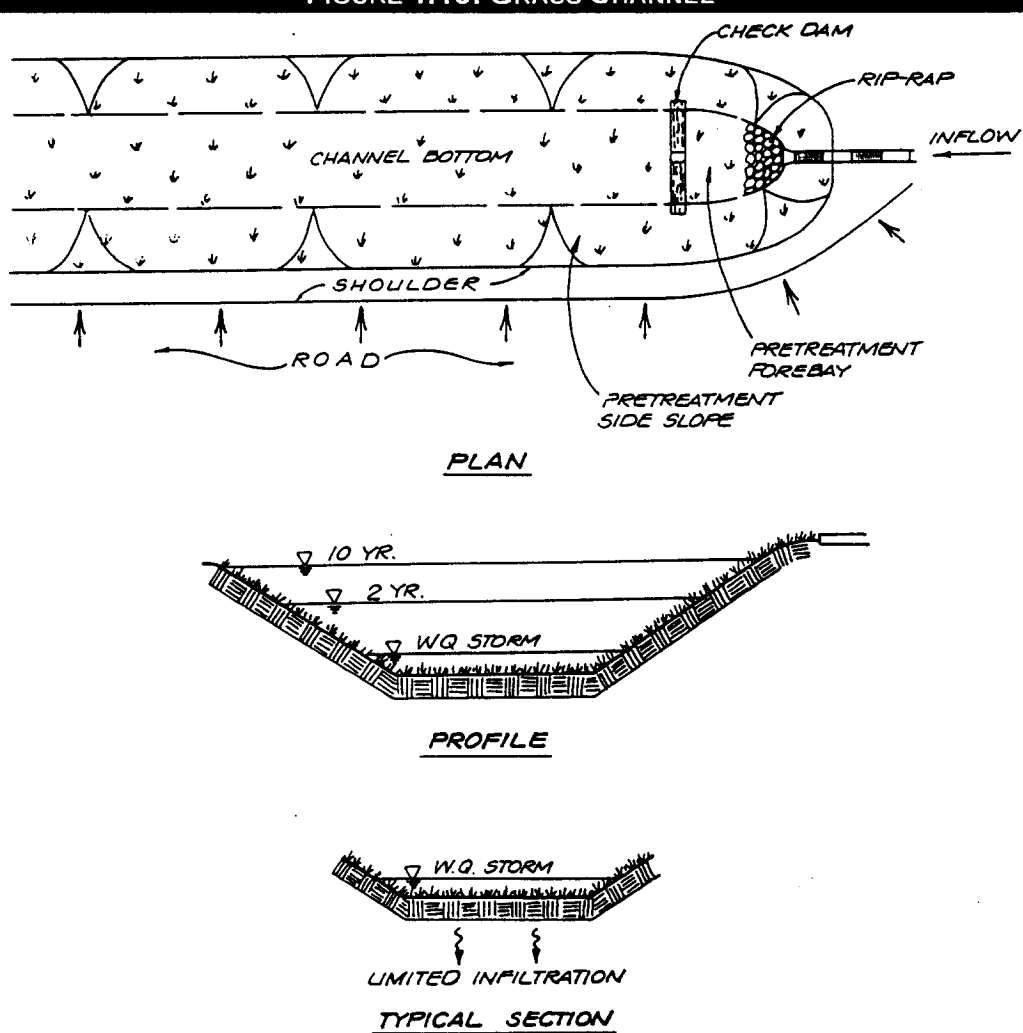
FIGURE 1.9: OPEN VEGETATED CHANNELS



The open channel design in most common use is termed a drainage channel, and is designed to have enough capacity to safely convey runoff during large storm events without erosion. Typically, a drainage channel has a cross-section with hydraulic capacity to handle the peak discharge rate for the ten year storm event, and channel dimensions (i.e., slope and bottom width) that will not exceed a critical erosive velocity during the peak discharge rate associated with the two year storm event (Figure 1.9a). Consequently, most drainage channels provide very limited pollutant removal, unless soils are extremely sandy and slopes are very gentle.

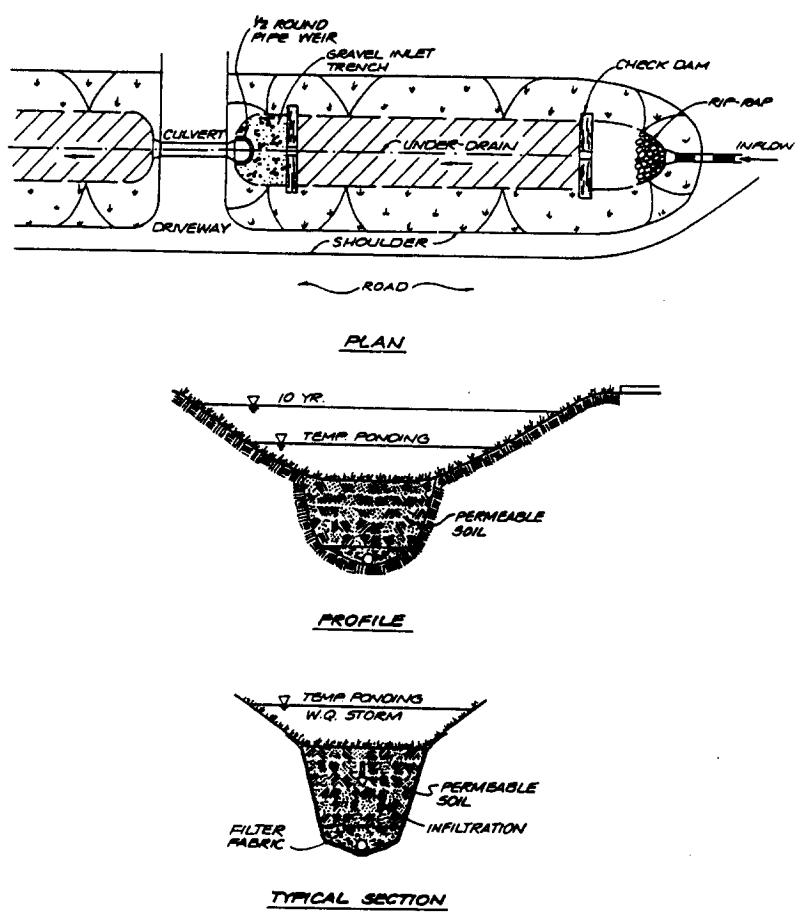
To achieve greater pollutant removal, stormwater engineers have recently employed *grass channels*. Grass channels are designed to meet runoff velocity targets for two very different storm conditions—a water quality design storm and the two year design storm (Figure 1.10). During the "water quality storm," runoff velocity typically cannot exceed 1.0 fps during the peak discharge associated with the water quality design rainfall event, and the total length of the channel must provide at least ten minutes residence time. In some regions of the country, grass channels are termed "biofilters" (Seattle METRO, 1992). To meet the water quality criteria, grass channels must have broader bottoms, lower slopes and denser vegetation than most drainage channels. Nominal pretreatment is created by placing checkdams across the channel below pipe inflows, and at various other points along the channel. The filter bed area in a grass channel is usually confined to the top inch of soil and thatch, since most runoff events will traverse the length of channel in about ten minutes.

FIGURE 1.10: GRASS CHANNEL



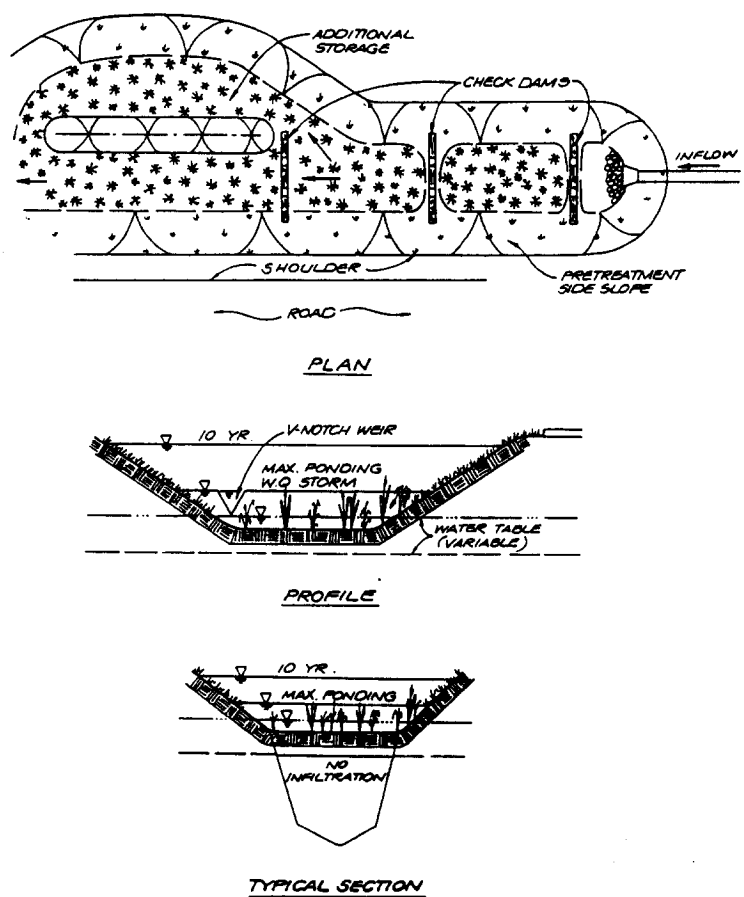
A third open channel is termed the dry swale. In a dry swale, the entire water quality volume is temporarily retained by checkdams during each storm. Unlike the grass channel, the filter bed in the swale consists of 30 inches of prepared soil (sandy loam) that is then collected by an underdrain pipe (see Figure 1.11). The swale is designed to rapidly dewater, thereby allowing front yards to be more easily mowed. Again, pretreatment is provided through check dams at pipe inflow points, and by keeping side slopes gentle if they are adjacent to impervious areas. In the event that surface soils clog, the dry swale has a pea gravel window on the downstream side of each checkdam to route water to the underdrain. A dry swale is often the preferred open channel option in residential settings since it is designed to prevent standing water that makes mowing difficult and generates complaints.

FIGURE 1.11: DRY SWALE



The last open channel design is termed a wet swale, and occurs when the water table is located very close to surface (Figure 1.12). As a result, swale soils often become fully saturated, or have standing water all or part of the year after the channel has been excavated. This "wet swale" essentially acts as a very long and linear shallow wetland treatment system. Like the dry swale, the entire water quality treatment volume is stored and retained within a series of cells in the channel, formed by berms or checkdams. The notched checkdams are set so that the invert creates the pool level when the water table is high. The dimensions of the notches are set to provide the desired detention time within each cell for the storm. In some cases, the cells may be planted with emergent wetland plant species to improve removal rates. If land is available, some wetland cells can be placed off-line, as shown in Figure 1.12.

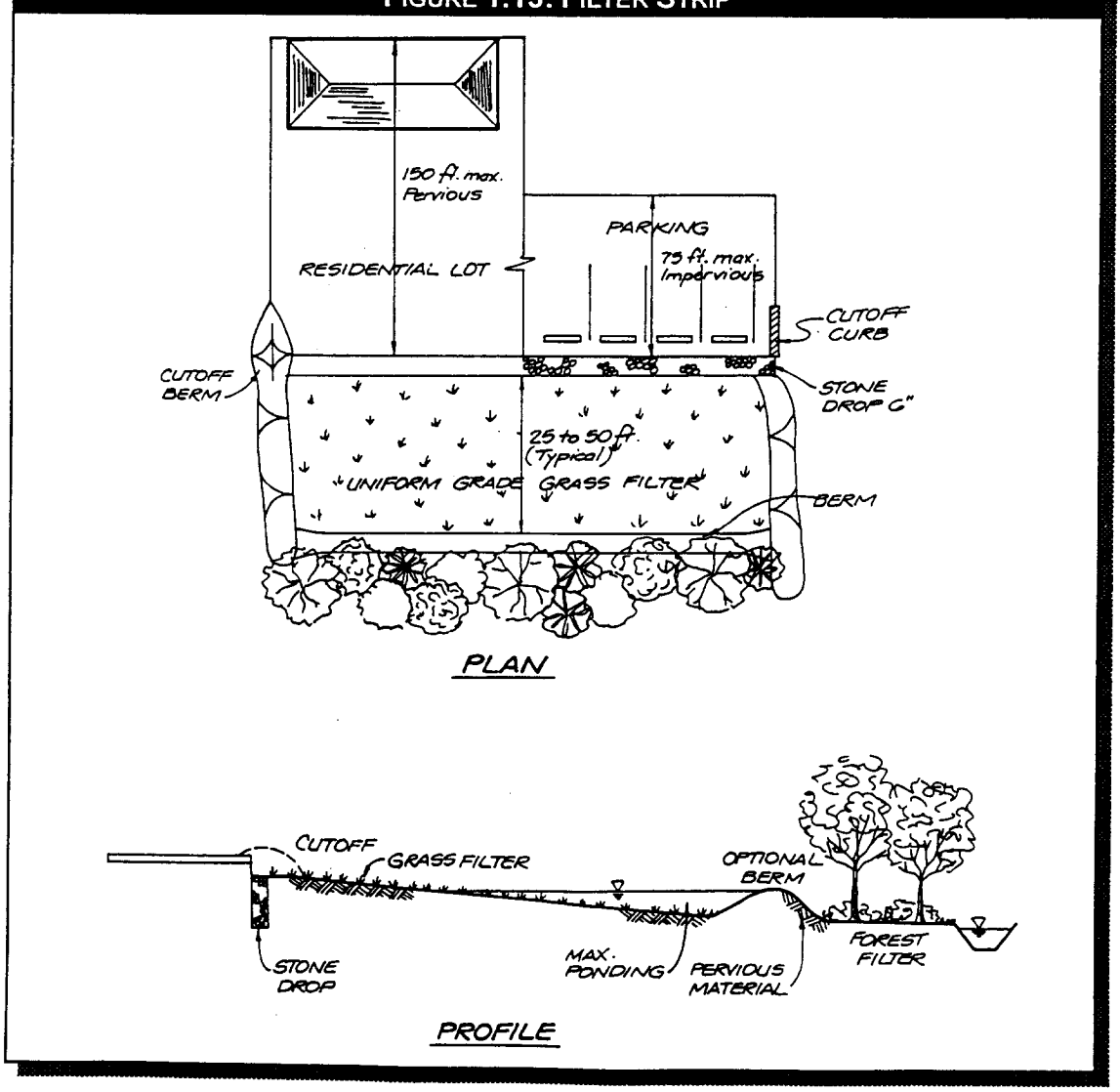
FIGURE 1.12: WET SWALE



1.3D FILTER STRIP

Filter strips rely on the use of vegetation to slow runoff velocities and filter out sediment and other pollutants from urban stormwater. To be effective, however, filter strips require the presence of sheet flow across the entire strip. Once flow concentrates to form a channel, it effectively short-circuits the filter strip. Unfortunately, this usually occurs within a short distance in urban areas. It is doubtful, for example, whether sheetflow can be maintained over a distance of 150 feet for pervious areas, and 75 feet for impervious areas (or about one parking bay). In the most common design, runoff is directed from a parking lot into a long filtering system composed of a stone trench, a grass strip and a longer wooded strip (see Figure 1.13).

FIGURE 1.13: FILTER STRIP

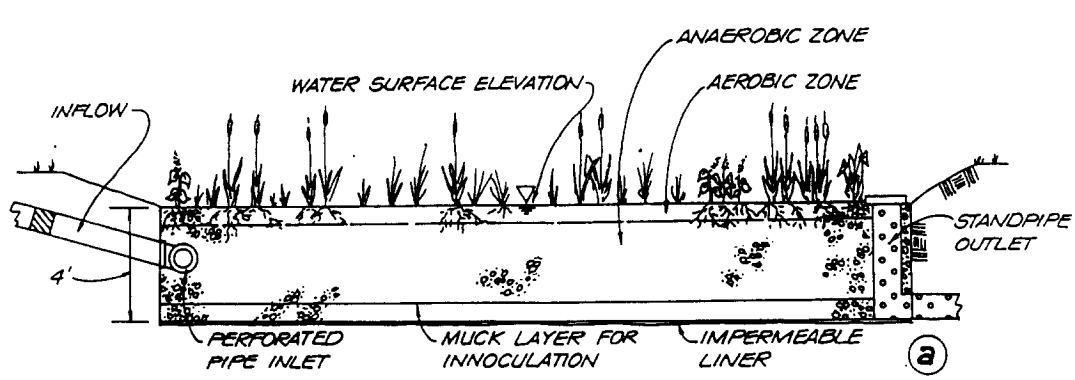


The grass portion of the filter strip provides pretreatment for the wooded portion. In addition, a six inch stone drop is located at the edge of the parking lot and the filter strip to prevent sediments from depositing at this critical entry point. The filter strip is typically an on-line practice, so it must be designed to withstand the full range of storm events without eroding (i.e., up to the peak discharge associated with the 100 year design storm). In snowier climates, the grass portion of the system provides a handy location to stockpile snow where the meltwater can gradually infiltrate into the soil. The maintenance requirements include scraping the sediment buildup at the edge of the parking lot to maintain inflows, and mowing the grass portion of the filter strip.

1.3E SUBMERGED GRAVEL FILTER

A recent design innovation is the submerged gravel filter. It consists of a series of cells that are filled with crushed rock or gravel (Figure 1.14). The standpipe from each cell is set at an elevation that keeps the rock or gravel submerged. Wetland plants are rooted in the media, where they can directly take up pollutants. In addition, algae and microbes thrive on the enhanced surface area of the rocks. In particular, the anaerobic conditions on the bottom of the filter can foster the denitrification process (Kadle and Knight, 1996). Although widely used for wastewater treatment in recent years, only a handful of submerged gravel filters have been designed to treat stormwater. In general, the submerged gravel filter has similar design components to the pocket sand filter.

FIGURE 1.14: SUBMERGED GRAVEL FILTER



1.4 A UNIFIED DESIGN APPROACH

The remainder of the manual presents detailed engineering guidance on each of the first four groups of filtering systems. Some unique features of the manual include:

1.4A A UNIFIED DESIGN APPROACH

The underlying concept of the manual is that a common and unified approach is needed to design each type of stormwater filter, so that this useful technology can gain wider engineering acceptance at the local level throughout the Chesapeake Bay.

1.4B SMALL STORM HYDROLOGY AND STORMWATER HOTSPOTS

A key feature of the manual is the presentation of methods to determine the hydrologic response and pollutant loading from small storms for smaller sites (Chapter 2). Small sites are not always the same, and can often be best modeled with new techniques for calculating runoff rates and volumes that reflect small storm hydrology from small, heterogeneous urban sites. Field research has indicated these methods are superior to the conventional NRCS runoff forecasting methods (such as TR-55 and TR-20) on small sites.

1.4C VOLUME-BASED SIZING

The manual presents a single volumetric sizing requirement for each filter which is to capture and treat 90% of the runoff producing rainfall events that occur each year. Many prior design approaches had been rate-based, and resulted in limited and unreliable pollutant removal rates.

1.4D FILTER SELECTION CRITERIA

What is the most appropriate stormwater filter for a particular development site? Are other BMP systems such as ponds, wetlands or infiltration more effective or appropriate? To answer these questions the manual synthesizes recent research and field experience on the pollutant removal performance, longevity, cost, and maintenance burden of each type of stormwater filter. This information has been condensed in a series of tables in Chapter 3 that help designers and municipal officials select the most effective stormwater filter for their development situation, and compare it against the performance and feasibility of other stormwater BMP options.

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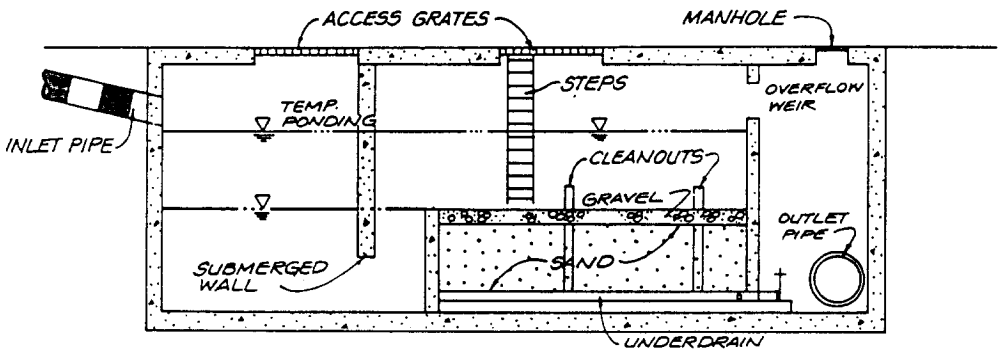
1.4E REVIEW OF POLLUTANT REMOVAL PATHWAYS

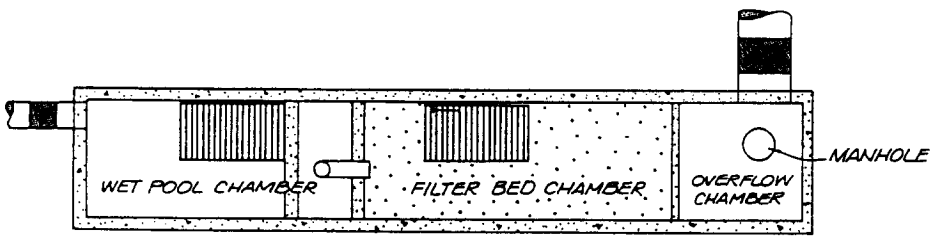
The latest performance monitoring data for stormwater filtering systems is reviewed in Chapter 4 to identify key pollutant removal pathways that can be enhanced in design. Both practical and innovative techniques for enhancing pollutant removal in each group of filter practices are recommended.

1.4F STANDARD DESIGN FEATURES AND DESIGN EXAMPLES

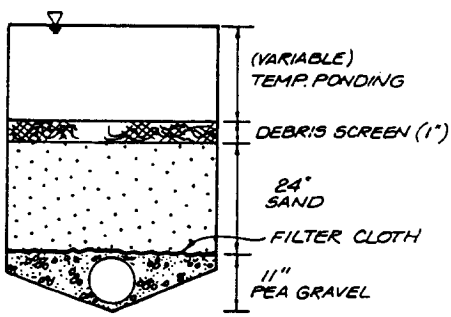
Chapter 5 presents detailed engineering design guidance for sand filters. The design of bioretention systems is presented in Chapter 6. Open channel systems and filter strip design are outlined in Chapter 7. Each design chapter outlines the basic filter sizing criteria, and incorporates standard engineering specifications for flow regulation, pretreatment, filter bed and media, and outflow mechanisms. This standardization should increase the effectiveness of each filtering practice and reduce maintenance problems. In addition, step-by-step design examples are presented for most practices that walk the engineer through the design methods.



PROFILE



PLAN



TYPICAL SECTION

Chapter

2

Runoff and Water Quality Characteristics of Small Sites

CHAPTER 2
RUNOFF AND WATER QUALITY CHARACTERISTICS OF SMALL SITES

In this chapter, we explore the hydrology and pollutant loading dynamics of small sites, where stormwater filtering systems are typically applied. Such understanding about the quality and quantity of runoff generated from small sites is essential to design effective stormwater filtering systems. Small sites are not homogeneous, and their hydrologic and water quality characteristics can be very different. For example, a site may exhibit a sharply different runoff response depending whether impervious areas are directly or indirectly connected to the storm drain system. Other source areas demonstrate different abilities to accumulate pollutants, or are heavily influenced by a unique pollutant loading source (industry, vehicles, pets, fertilizer). In other cases, certain impervious surfaces themselves may actually produce greater pollutant loadings (e.g, leaching of zinc from rooftops). Even pervious areas, such as lawns, can exhibit different hydrologic and water quality responses, depending on the degree of soil compaction or lawn management.

To better understand the hydrology and pollutant loading dynamics of small sites, the chapter is divided into six sections.

The first section explores the "average" concentration of pollutants in stormwater runoff for a range of different urban source areas, based on a synthesis of small site monitoring research. This section also examines specific stormwater "hotspots" that have the potential to generate greater loads of hydrocarbons, metals and priority pollutants, and therefore may warrant greater treatment and/or a higher level of groundwater protection.

The second section presents a simple method to break down small sites into individual source area units that have distinct hydrologic or water quality properties.

The third section investigates the unique hydrology during small storms for urban source areas. Recent research suggests that the hydrologic response of a small site can differ greatly depending on the nature of a source area. A simple adaptation of the NRCS stormwater peak discharge model (TR-55) is proposed to get more accurate runoff predictions from small sites.

The concept of the rainfall frequency spectrum is presented in the fourth and fifth sections. This simple rainfall analysis technique is a powerful tool for determining how much rainfall can be treated in a stormwater filter, and how much must be bypassed.

A unified method for defining and deriving the water quality volume and flow rate is advanced in the last three sections. This standard method is then used as the basis for sizing all stormwater filters considered in this manual.

R0021985

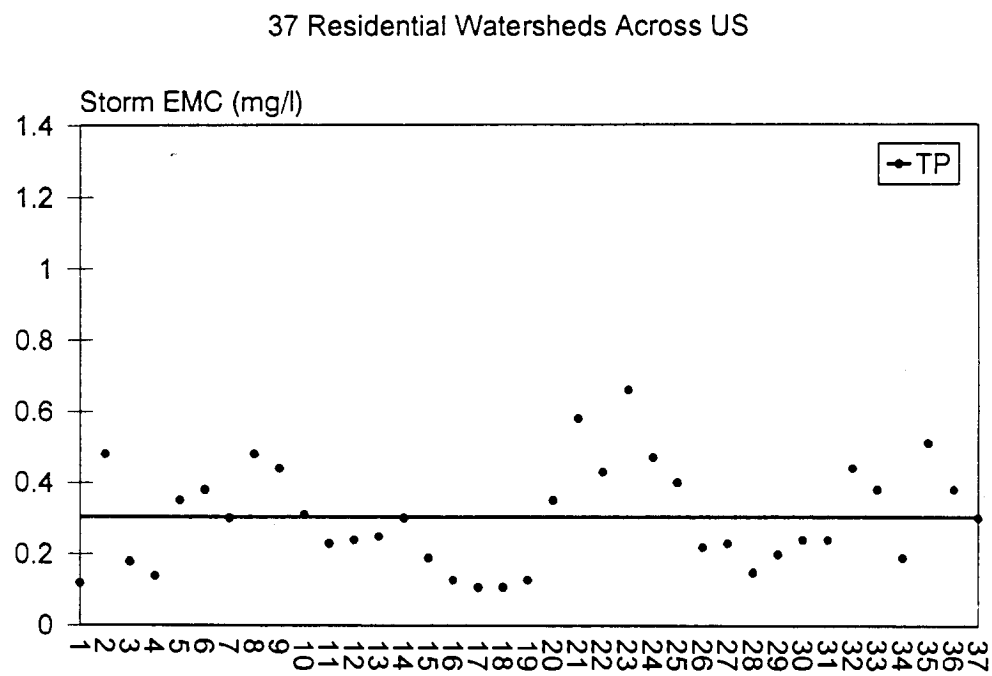
2.1 COMPARISON OF STORMWATER QUALITY FROM DIFFERENT SOURCE AREAS

One of the conclusions of the massive national EPA NURP monitoring study was that while pollutant concentrations were indeed variable at each site, there appeared to be no statistical difference among commercial, industrial and residential land uses at the catchment level (25 to 500 acres). In general, mean pollutant concentrations found in stormwater runoff were surprisingly consistent at the catchment or watershed level (see Table 2.1). One example of this consistency is the mean phosphorus concentration observed in stormwater runoff at 37 different catchments across the U.S. with widely different climate, soils, density and vegetative cover (Figure 2.1). Despite such differences, the average concentration of total phosphorus is about the same no matter where the runoff was sampled.

TABLE 2.1: MEAN POLLUTANT CONCENTRATIONS (IN MG/L) FOR SELECTIVE PARAMETERS FOR STORMWATER RUNOFF (SOURCE: EPA, 1983)

Pollutant	*New Suburban NURP Sites (Washington, DC)*	*National NURP Study Average*
Phosphorus		
Total	0.26	0.46
Ortho	0.12	-
Nitrogen		
Total	2.00	3.31
Nitrate	0.48	0.96
TKN	1.51	2.35
COD	35.6	90.8
BOD (5-Day)	05.1	11.9
Metals		
Zinc	0.037	0.176
Lead	0.018	0.180
Copper	-	0.047

FIGURE 2.1: TOTAL PHOSPHORUS CONCENTRATION IN STORMWATER RUNOFF



2.1A ATMOSPHERIC DEPOSITION AS THE PRIMARY POLLUTANT SOURCE

The primary explanation for the observed consistency is that the primary source of pollution was more or less the same regionally and nationally—atmospheric deposition. Thus, the basic model is that pollutants are deposited from the atmosphere as either dryfall or wetfall where they accumulate on impervious surfaces. The stored pollutants are subsequently washed off during storm events. Monitoring of the deposition rates for common pollutants found in stormwater runoff does indeed suggest that atmospheric deposition is the dominant source of many pollutants found in stormwater runoff. This is evident in the summary of Washington metropolitan area deposition rates provided in Table 2.2. As can be seen, atmospheric deposition alone can account for most, if not all, of the observed concentrations of total nitrogen, zinc and several other pollutants. While atmospheric deposition provides the base loading for many pollutants, other sources can significantly add to the overall pollutant loading for a site. These urban source areas are described in the next section.

TABLE 2.2: AVERAGE ANNUAL ATMOSPHERIC POLLUTANT DEPOSITION IN THE WASHINGTON METROPOLITAN AREA - LBS/AC/YR (ADAPTED FROM MWCOG, 1983)

Pollutant	Suburban	Urban	Dryfall or Wetfall
Solids	155	243	D>>W
Chemical Oxygen Demand	133	210	W>D
Total Nitrogen	1.8	17	W=D
Organic Nitrogen	7.2	10.2	W>D
Nitrate-Nitrogen	5.6	6.8	W>>D
Total-Phosphorus	0.5	0.8	D>W
Ortho-Phosphorus	0.26	0.35	D>W
Copper	0.21	0.61	W>>D
Cadmium	0.09	0.003	ND
Lead	0.44	0.53	D>>W
Nickel	0.56	0.08	ND
Zinc	1.35	0.65	W>>D

2.1B URBAN SOURCE AREAS

Monitoring of individual urban source areas is a relatively new line of research. Instead of monitoring an urban catchment that has many source areas that contribute to the observed pollutant concentration, researchers focus their monitoring on a single source area, such as a rooftop, parking lot, street, or lawn, to determine the range and concentration of pollutants that it produces. Are pollutant levels higher or lower than the national or regional average? Does the particular source area frequently produce hydrocarbons, metals or toxics that are not commonly found in urban runoff?

To answer these questions, mean pollutant concentrations were computed for 15 individual source areas, based on 20 published research studies conducted from a variety of geographic areas. As with any compilation derived from such diverse data sources, several caveats should be kept in mind in interpreting them. For example, the mean concentrations used to characterize a given source area

represent a group mean averaged over one to nine monitoring sites. The sample size for individual studies ranged from two to 200 runoff samples. Monitoring was performed in all regions of the country, and different sampling methods were employed by different researchers. Characteristics of individual source areas (e.g., commercial parking lots) were not frequently reported; this prevents an exact comparison between groups. Literature sources and the group means are provided in Appendix A. Thus, mean stormwater concentrations for each of the 15 source areas should be considered very provisional, and subject to further change and refinement as more data is acquired.

2.1c STORMWATER HOTSPOTS

Stormwater hotspots are defined as a land use or activity that generates higher concentrations of hydrocarbons, trace metals or toxicants than are found in typical stormwater runoff, based on monitoring studies. The increased pollutant loadings from these hotspots can generate concerns about the risk of groundwater contamination and or the toxicity in sediments or the water column of surface waters. If a site is deemed a stormwater hotspot, it has two important implications. First, a higher or more effective form of stormwater treatment is required to remove the elevated concentration of pollutants. Second, treated runoff from a BMP must be prevented from infiltrating into groundwater.

Designers should assess their sites to determine if any potential hotspots are present. It should be kept in mind that not all urban land uses or activities have been fully monitored to characterize the quality of their stormwater. Based on recent monitoring, however, a number of sites do appear to have hotspot characteristics. A preliminary list of potential stormwater hotspots includes the following land uses or activities:

- airport deicing facilities
- auto recycler facilities
- commercial nurseries
- commercial parking lots
- fueling stations
- fleet storage areas (bus, truck)
- industrial rooftops (depending on the nature of the roof surface)
- marinas
- outdoor container storage of liquids
- outdoor loading/unloading facilities
- public works storage areas
- SARA 312 generators (only if materials or containers are exposed to rainfall)
- vehicle service and maintenance areas
- vehicle and equipment washing/stream cleaning facilities

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While it would be tempting to classify any industrial facility as a potential stormwater hotspot, in reality, these sites can range from very clean to very dirty, depending on the industrial process, site conditions, and the nature of chemical inputs. The key point is to analyze the site to see if rainfall comes into contact with materials or surfaces, and has the potential for subsequent washoff. Many industrial facilities are required to have a NPDES stormwater discharge permit and pollution prevention plan, under the Clean Water Act. As part of their EPA stormwater NPDES permit, many individual facilities have collected monitoring data to characterize the quality of their stormwater runoff. This data can be very helpful in determining whether an industrial site has the potential to become a stormwater hotspot.

At the same time, there are many land uses and activities that are not normally considered to have the potential to create a hotspot. Runoff quality from these areas is comparatively low with respect to hydrocarbons, metals and pollutants. These include:

- streets and highways
- residential developments
- institutional developments
- office developments
- non-industrial rooftops
- pervious areas

2.2 BREAKING DOWN A SITE INTO SOURCE AREAS

Small sites are not always homogenous in urban and suburban areas. In broad terms, every site can be broken down to those areas that are **pervious** to rainfall and those that are **impervious** to it (and therefore create more runoff). Within these two broad categories, however, there is quite a bit of variation in hydrological response and pollutant dynamics.

2.2A TYPES OF IMPERVIOUS COVER

Impervious areas can be further broken down into five subcategories.

ROOFTOP SOURCE AREAS

Rooftop source areas often tend to produce cleaner runoff given their elevation and pitch. Usually, the only major source of pollutants is atmospheric deposition. In some cases, however, acid rainfall can leach or desorb pollutants contained within the rooftop surface. This effect is best seen when the rooftop source area monitoring data are grouped into *residential*, *commercial* and *industrial* surfaces (Table 2.3). As can be seen, sediment and phosphorus levels are both well below mean national stormwater concentrations. Bacterial levels are also comparatively low, presumably caused only by birds. On the other hand, rooftop runoff often

contains higher metal concentrations than many source areas, especially copper and zinc. The source of the metals appear to be from leaching of painted roof surfaces, galvanized gutters and downspouts, or copper flashing. In general, the effect is modest for residential rooftop runoff (about twice the national average), but is much more pronounced in commercial and industrial rooftops.

TABLE 2.3: SOURCE AREA MONITORING SUMMARY - ROOFTOP RUNOFF

Parameter	*Residential*	*Commercial*	*Industrial*
TSS (100 mg/l)	19	9	17
Total P (0.3 mg/l)	0.09	0.20	0.08
f. Coliforms (c/100ml)	2600	1100	5800
Copper (10 ug/l)	20	7	62
Zinc (160 ug/l)	312	256	1390

PARKING LOTS

Parking lots are a fairly diverse source area, depending on their size, vehicle turnover, age and land use. This source area is strongly influenced by emissions and leakage from vehicles, as well as atmospheric deposition. Consequently, parking lot runoff tends to have greater concentrations of trace metals than the national stormwater mean (e.g, cadmium, copper, lead and zinc) as well as oil and grease (Table 2.4). In addition, monitoring of parking lots results in a greater number of detections of priority pollutants. Some variation is seen between parking lots that serve *industrial* versus *commercial* land uses. In many cases, industrial parking lots may also contain stored materials, loading docks, parked equipment or fueling/ service areas that may be pollutant sources.

TABLE 2.4
SOURCE AREA MONITORING SUMMARY - PARKING LOTS

Parameter	Commercial	Industrial
T.S. (100 mg/l)	27	228
Cadmium (0.5 up/l)	8	2
Copper (10 up/l)	51	34
Lead (18 up/l)	28	85
Zinc (160 up/l)	139	224
Oil/Grease (1-2 mg/l)þ	8.5	15.0

STREETS AND HIGHWAYS

As might be expected, pollutant concentrations in road runoff are also heavily influenced by vehicles, and tend to increase as traffic volume increases (Bannerman, 1994). *Urban highways* carry the greatest average daily traffic volume and tend to have sediment, organic carbon, nutrient and metal levels that are about twice the national stormwater mean (Table 2.5). Streets exhibit a similar trend, but also appear to be influenced by the land use that they serve. Table 2.6 shows sediment, metal and oil/grease levels for commercial and residential streets. In general, *commercial streets* appear to be a potent source area for many stormwater pollutants, which may reflect traffic volume, poor upkeep of shoulders, sanding or other factors. These include cadmium, copper, zinc and priority pollutant detection. Runoff from *residential streets*, on the other hand, is about two to five times lower for sediment, metals, and hydrocarbons, and is fairly close to the national stormwater mean. Notable exceptions in residential street runoff are very high bacteria and nutrient levels, which are presumably due to pets and blow-in from adjacent pervious areas.

TABLE 2.5: COMPARISON OF MEDIAN EVENT MEAN CONCENTRATION AT HIGHWAY AND NURP RUNOFF SITES (mg/l) (SOURCE: FEDERAL HIGHWAY ADMINISTRATION, 1990)

	Urban Highway Sites	*NURP Mixed Sites (Come & Res)*	*Ratio of Highway/ NURP Mixed Sites*
TSS	142	67	2.1
COD	114	65	1.8
TKN	1.83	1.29	1.4
PO4-P	0.40	0.26	1.5
Copper	0.054	0.027	2.0
Lead	0.400	0.114	3.5
Zinc	0.329	10.154	2.1
		Average	2.0

TABLE 2.6: SOURCE AREA MONITORING SUMMARY - STREETS AND HIGHWAYS

Parameter	*Residential Street*	*Commercial Street*	*Urban Highway*
TSS (100 mg/l)	172	468	142
Cadmium (0.5 ug/l)	1.0	6.7	1.0
Copper (10 ug/l)	25	73	54
Lead (18 ug/l)	51	170	410
Zinc (160 ug/l)	173	450	329
Oil/Grease (1-2 mg/l)	2.0	3.7	ND

AUTOMOTIVE

Areas where vehicles are fueled, serviced or disposed can represent a significant source area of many stormwater pollutants. Table 2.7 shows reported stormwater concentrations for three automotive areas: *car service or fueling stations*, *auto recyclers* and *auto manufacturing*. In each case, the concentration of cadmium, copper, zinc, oil and grease and priority pollutant detections is among the highest reported for any urban source area. The primary reason is the higher risk that automotive fluids will spill, leak or drip onto impervious surfaces, or that other automotive parts (such as batteries, brake linings, paint, and metal dust) will come into contact with rainwater. Higher stormwater concentrations are often observed when wrecked cars are present, or where cars are scrapped.

TABLE 2.7: SOURCE AREA MONITORING SUMMARY - AUTOMOTIVE

Parameter	*Vehicle Service/Fuel*	*Recycler*	*Industrial*
TSS (100 mg/l)	31	355	124
Cadmium (0.5 ug/l)	9	8.5	ND
Copper (10 ug/l)	88	103	148
Lead (18 ug/l)	80	182	290
Zinc (160 ug/l)	290	520	1600
Oil/Grease (1-2 mg/l)	14	25	ND

RESIDENTIAL (NON-ROOFTOP)

The last major urban source area is residential areas, and includes *lawns* and *driveways*. In general, pollutant concentrations in residential runoff are relatively innocuous, and are usually well below the national stormwater mean for most metals, hydrocarbons and priority pollutants (see Appendix A Stormwater Pollutant Concentrations from Different Source Areas and Hotspots). Runoff from lawn and driveways, however, ranks among the highest concentration of several conventional stormwater pollutants, including sediment, total phosphorus (Table 2.8) and bacteria (Table 2.9). The importance of lawns as a source area is tempered, however by the fact that they generate relatively low runoff volumes, in comparison to impervious surfaces, and therefore produce a smaller annual mass loading of sediment and nutrients than other urban source areas (Bannerman, 1994).

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TABLE 2.8: TOTAL PHOSPHORUS SOURCE AREAS IN THE URBAN LANDSCAPE

Source Area	*Concentration (mg/l)*
Rooftops	0.11
Commercial Parking Lot	0.45
Industrial Parking Lot	0.65
Residential Street	0.63
Commercial Street	0.47
Urban Highway	0.40
Lawns	1.67
Driveway	1.16
Snowbelt	0.70
Typical Stormwater	0.30

TABLE 2.9: FECAL COLIFORM BACTERIA SOURCE AREAS

Source Area	*Concentration (counts/100 ml)*
Rooftops	2,400
Parking Lots	2,500
Residential Streets	37,000
Commercial Streets	12,000
Lawns	24,000
Driveways	34,000

2.2B TYPES AND DISTRIBUTION OF PERVIOUS COVER

Pervious areas are a very diverse and complex mosaic of surfaces—forests, wetlands, meadows, lawns, turf, landscaping and the ubiquitous "vacant" lands. While the mix varies based on the history and intensity of past development, pervious cover can be grouped into one of five categories, depending on their vegetative cover and management (Figure 2.2). The estimated distribution of each type of pervious cover in a typical suburban landscape is shown in Figure 2.3.

FIGURE 2.2: TYPES OF PERVIOUS COVER

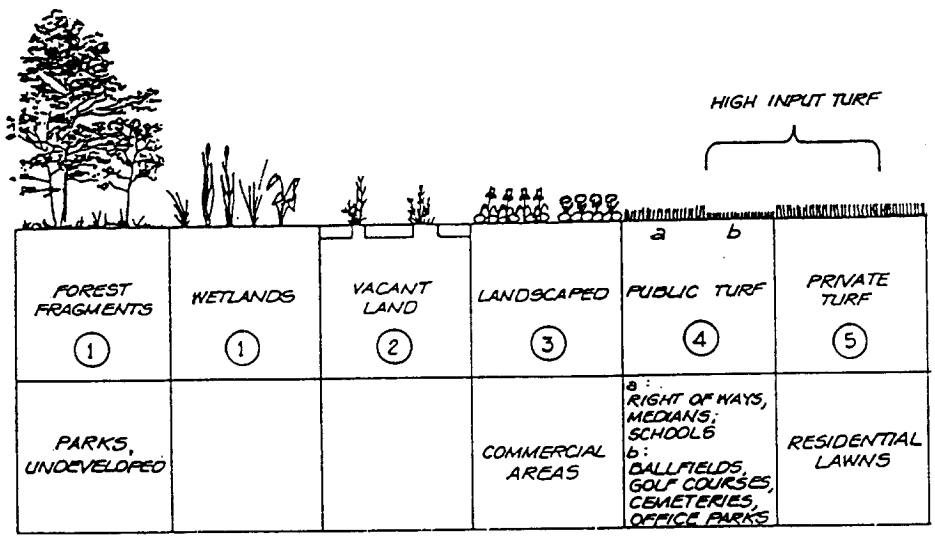
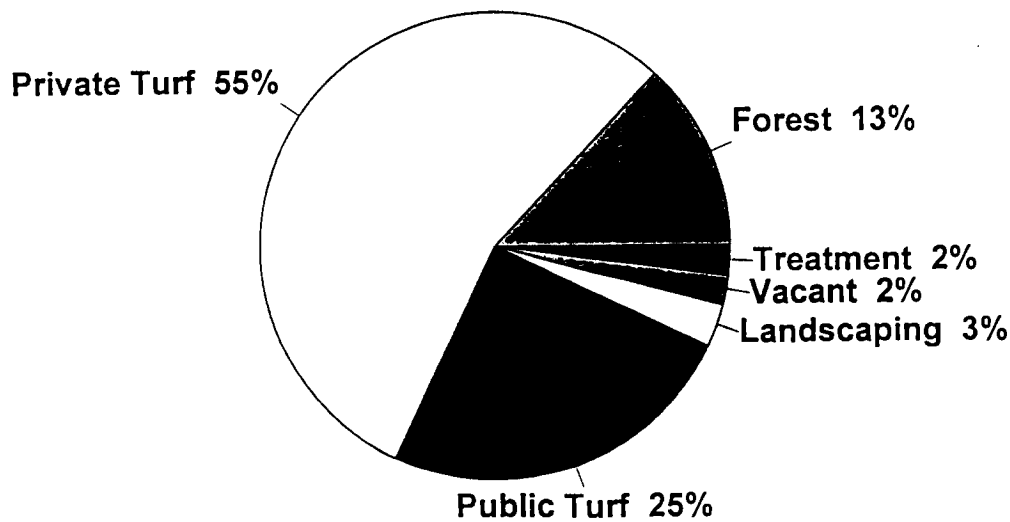


FIGURE 2.3: DISTRIBUTION OF PERVIOUS COVER



URBAN FORESTS AND WETLANDS

The extent of forests and wetlands in the urban landscape varies considerably from one region of the country to another, and even one city to another. The composition and diversity of the forest often changes remarkably due to urbanization, with a strong shift to non-native tree species and invasive shrubs and vines (Adams, 1994). As many as 30 to 60% of native forest species disappear from the highly urban forest community. Much of the forest cover in urban areas is often limited to isolated stands or individual street trees. While these small forest islands are important, they lack the structure, soils, and understory found in natural forests.

PRIVATE TURF (LAWNS)

The best estimate of the extent of home lawns is that they comprise about 70% of the total turf area of the urban landscape (Cockerham and Gibeault, 1985). The lawn category can be further subdivided into *high* and *low* input lawns. *High-input* lawns are defined as those that are regularly fertilized, irrigated and receive applications of herbicides or insecticides. Homeowners apply chemicals to roughly two-thirds of high-input lawns, while the remaining third is treated by lawn care companies. *Low-input* lawns are defined as those lawns that are regularly mowed, but seldom receive any chemical inputs. Surveys indicate that the percentage of high and low-input lawns are about equal in most urban areas.

PUBLIC TURF

About 30% of the remaining turf in urban areas is devoted to "public turf," which include parks, golf courses, schools, churches, cemeteries, median strips, utility corridors and office parks. The greatest share of public turf appears to be contained within parks, golf courses and school grounds. Management of public turf runs the gamut from regular mowing to very intensive turfgrass management (e.g., golf courses).

INTENSIVELY LANDSCAPED AREAS

Commercial areas can comprise up to 20% of the urban landscape. Although commercial areas are highly impervious, many localities require that 5 to 10% of the site be intensively landscaped to provide visual relief, shade and create a more attractive environment. Much of this landscaping is in small fragments that are graded to run onto adjacent impervious areas.

VACANT LANDS

Some portion of urban lands is always in transition from one use to another and remains vacant until that change occurs. In general, these vacant or open lands are temporary in nature and they receive little in the way of vegetative management (although they may be quickly invaded by invasive or pioneer species). Depending on how long the area has been vacant, the cover can range from bare earth, weeds,

meadow or shrubs. Erosion can be severe if vegetative cover is poor.

Each of the five types of pervious cover have been highly disturbed by man and lack many of the qualities associated with similar cover types located in natural areas. Perhaps the greatest single change relates to the disturbance of native soils. Development usually involves wholesale grading of the site, removal of topsoil, severe erosion during construction, compaction by heavy equipment, and filling of depressions. In recognition of this disturbance, most soil surveys change the native soil type to the ubiquitous moniker "urban soils" after a site is developed. Urban soils tend to be highly compacted, poor in structure and low in permeability. As a result, these soils often produce more runoff than before they were disturbed. For example, Pitt (1994) noted that one third of the disturbed urban soils he tested in Milwaukee had an infiltration rate of zero or near zero, exhibiting the same runoff response as concrete or asphalt.

2.2c THE EDGE EFFECT: RELATIONSHIP BETWEEN PERVIOUS AND IMPERVIOUS COVER

When seen from the air, most impervious areas are small islands interspersed in a sea of pervious cover, ranging from a few hundred square feet to a few acres in size. The urban landscape is a complex mosaic of pervious and impervious cover that are linked and interlaced together. Since many impervious areas are linear in form (e.g., roads, sidewalks, and parking lots), extensive edges are created between the two types of cover. We tend to think of pervious and impervious areas as distinct and separate. Indeed, most hydrological models simulate the hydrological and water quality response of each area independently. Given the close proximity to each other, the assumption that the two areas do not interact is questionable.

From a hydrological perspective, pervious cover can only be understood in relation to its adjacent impervious cover. More precisely, if the direction of flow is from pervious cover to impervious cover, then the stormwater will occur as *runoff*. On the other hand, if water flows from impervious cover to pervious cover, then the stormwater will occur as *runon*, and is much more likely to infiltrate into the soil. The practical implication is that if a site is graded to produce runon, it may be possible to significantly reduce the volume of stormwater runoff (see section 2.7). Under some conditions, it may be possible to reduce stormwater pollutant loads, as well.

Some examples include directing rooftop runoff to travel through downspouts and over grassed yards, road runoff into swales rather than curb and gutters, and parking lot runoff to drain to forests or fields. The hydrologic effect of disconnecting impervious areas can be very significant, particularly in low-density residential watersheds. In some cases, disconnecting these impervious areas can create enough runon to reduce the "effective" impervious cover in a watershed by 20 to 50% (Sutherland, 1995).

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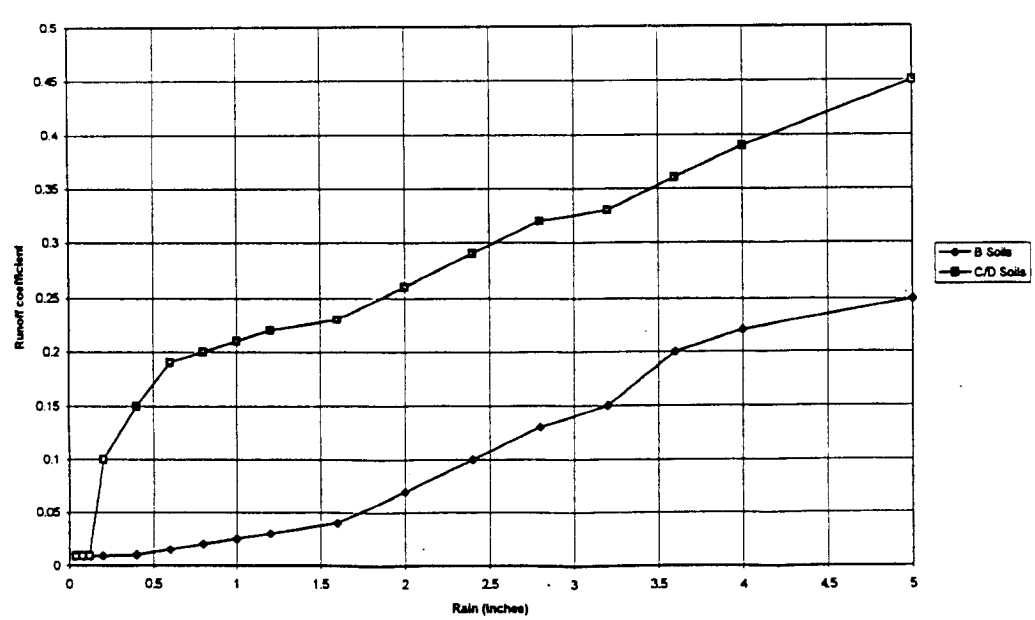
RUNOFF FROM PERVIOUS AREAS

While every effort should be made to maximize runon to pervious areas, drainage considerations often dictate that most pervious areas will still be graded to drain to impervious areas or storm drain collection systems. Consequently, the hydrologic response of each of the five types of pervious cover is of great interest. Most hydrologic research, however, has lumped all the types of pervious cover into a single category, or has assumed that pervious cover has the same properties as well tended turfgrass. Thus, the majority of urban hydrology models utilize the SCS curve number approach, where the runoff rate is dependent primarily on the soil type and to a lesser extent the vegetative cover at a site.

While these models have proven effective for predicting runoff volumes from pervious areas during larger storm events (3 to 5 inches or more), the curve number approach tends to grossly over-predict the runoff volumes produced during the smaller but more common events (Pitt, 1994). The small storm hydrology data presented by Pitt for two test watersheds (Figure 2.4) illustrate the increased runoff properties of urban lawns, presumably due to soil compaction. The volumetric runoff coefficients at these sites tended to progressively increase with rainfall volume, and were in the 0.10 to 0.23 range for soils in the "D" hydrologic soil group for moderate storm events. Lawns that had more permeable soils (in the "B" soil group) produced less runoff volume (Rv's ranging from 0.01 to 0.04 for small to moderate sized storms). Clearly, lawns may produce greater runoff volume then has been traditionally assumed. Even runoff testing of well-tended turfgrass has revealed that turfgrass still produces about half the runoff of bare soil during larger storms (McLean, 1995).

FIGURE 2.4: RUNOFF COEFFICIENT FOR PERVIOUS URBAN AREAS



PARTICLE SIZE DISTRIBUTION

One additional important aspect of stormwater runoff from different source areas is the relationship of particle size to pollutant load. Work done by Sartor and Boyd (1974) and Pitt (1987) starting in the early 1970's suggests that most of the total particulate load from urban runoff is made up by the coarser fractions, consisting of sand/gravel particle sizes greater than approximately 40 microns. Shaver and Baldwin (1991) reported that while nearly 94% of the urban runoff particulate load is from these coarser grained fractions, more than half of the phosphorus load and significant percentages of other pollutants are associated with fine grained silts and clays.

Particle size distribution is an important consideration for sizing the sedimentation chamber of a filter system. Shaver and Baldwin (1991) and Bell et al. (1995) specify that sand filters should only be used to treat runoff from impervious, or nearly-impervious surfaces. They argue that the larger percentage of particulates from impervious surfaces are in the coarser fractions, and therefore, filtering systems will be less prone to clogging. The logic follows that the sedimentation chamber will capture the coarser grained material, and the filter chamber will capture and treat the relatively small amount of finer grained material. Therefore, filters designed to treat runoff from purely impervious surfaces require less sedimentation area and volume than those designed to treat runoff from more pervious surfaces.

The City of Austin (1988) allows the use of sand filters for a range of land uses and drainage areas. They use a smaller, silt size particle (20 microns) as the target for sizing the sedimentation chamber, probably recognizing that more pervious areas are likely to contribute more fine grained particles In order to quantify and resolve the apparent discrepancy between the above criteria, this manual recommends that for drainage areas less than 75% impervious, the target particle size for designing the sedimentation chamber be set at 20 microns. For drainage areas with imperviousness greater than 75%, the target particle size should be set at 40 microns. See Chapter 5 for discussion and application of these sizing principles.

2.3 SMALL STORM HYDROLOGY

Small storms are responsible for most annual urban runoff and likewise are responsible for most pollutant washoff from urban surfaces. Therefore, the small storms are of most concern for water quality resource protection.

Large storms occur infrequently, and although they may contain significant pollutant loads (Chang, G., et al., 1990), their contribution to the annual average pollutant load is really quite small (due to the infrequency of their occurrence). In addition, there are longer periods of recovery available to receiving waters between larger storm events

allowing systems to flush themselves and the aquatic environment to recover.

The runoff **volume** is the most important hydrologic variable for water quality protection and design because water quality is a function of the capture and treatment of the mass load of pollutants. The runoff **peak rate** is the most important hydrologic variable for drainage system design and flooding analysis. Water quality facilities are designed to treat a specified quantity or volume of runoff for the full duration of a storm event as opposed to accommodating only an instantaneous peak at the most severe portion of a storm event.

To design effective BMPs and evaluate water quality impacts in urban watersheds, it is necessary to predict the amount of rainfall converted to runoff. The amount of rainfall which is converted to runoff is a function of storm characteristics such as rainfall amount, storm duration, rainfall intensity, and the urban land surface. These surfaces can be broken down into two main categories, pervious and impervious surfaces.

Impervious surfaces are traditionally thought to convert almost all rainfall into runoff, with pervious surfaces contributing much less runoff. In urban areas, particularly for small storms, this is not necessarily the case. Pervious surfaces can be heavily compacted and can have a surprisingly high runoff potential. Impervious surfaces, with minor cracks and expansion joints can have a remarkably high infiltration capability.

Impervious surfaces have five main components which contribute to rainfall losses:

- Interception of rainfall by over-hanging vegetation
- Flash evaporation
- Depression storage
- Sorption by dirt particles
- Infiltration through cracks and seams

The first four processes predominately occur immediately after the start of a rainfall event and dissipate within a relatively short time period and are therefore often referred to as initial abstractions. Infiltration through cracks and seams continues throughout the storm event and depending on the amount of rainfall, can account for significant losses. Many runoff models incorrectly estimate initial abstractions by holding them constant, and few consider infiltration through impervious surfaces for the duration of the storm event (Pitt, 1994).

The amount of runoff generated by pervious surfaces is related to the size of the pervious area, the relationship to impervious surfaces, the permeability of the underlying soils and the condition and type of vegetative cover.

The primary hydrologic methods to estimate storm runoff peak discharges in the Chesapeake Bay Watershed are the Rational Formula and SCS Methods, particularly, TR-55, "Urban Hydrology for Small Watersheds" (USDA, 1986). Several computer models, including SCS, TR-20, "Project Formulation, Hydrology" (USDA, 1982) and the U.S. Army Corps of Engineers', HEC-1 (U.S. Army, COE 1982) also utilize SCS methods to compute discharge rates. These methods are valuable for estimating peak discharge rates for large storms (i.e., >2") and larger drainage areas (> 10 to 25 acres), but can significantly underestimate the runoff from small storm events.

The limiting factors for the Rational Formula are in the computation of the time of concentration (usually set at a minimum of 5 minutes, which is hard to achieve on many small sites), the selection of "C" values for urban developments which do not address soil infiltration capability, and the equal weight placed on drainage area. The rational method is ideally suited for drainage design where peak rates of runoff are required, but does not estimate storm volume and therefore should not be used for water quality design.

Urban Hydrology For Small Watersheds (TR-55), as the title suggests, is recommended for urban watersheds with small drainage basins. This methodology has been used extensively for stormwater management design for quantity control (i.e., 2, 10, and 100 year management). TR-55 relies on a Curve Number (CN) instead of the "C" to reflect the percentage of rainfall converted to runoff. The TR-55 methodology also has the same limitations associated with computing the time of concentration for extremely small drainage areas.

One of the principal short comings of TR-55 is that the methodology assumes a constant CN for a large range of rainfall events. While this assumption does not significantly affect the accuracy of the model for larger storm events (> 2"), smaller rainfall events produce more runoff than are predicted by the SCS procedure (Pitt, 1994). This chapter presents a method for estimating the volume of runoff and peak discharge from small storms. Standard SCS methods should be used by designers for computing volumes and peak discharges for larger storm events (i.e., 2, 10 and 100 year storms).

Dr. Robert Pitt and his colleagues, have conducted several years of research on small storm hydrology, in several diverse geographic regions, over a wide range of land uses with remarkable consistency between simulated and observed results. The results of Pitt's research are described in Table 2.10.

TABLE 2.10: PRINCIPLES OF SMALL STORM HYDROLOGY (ADAPTED FROM PITT, 1994)

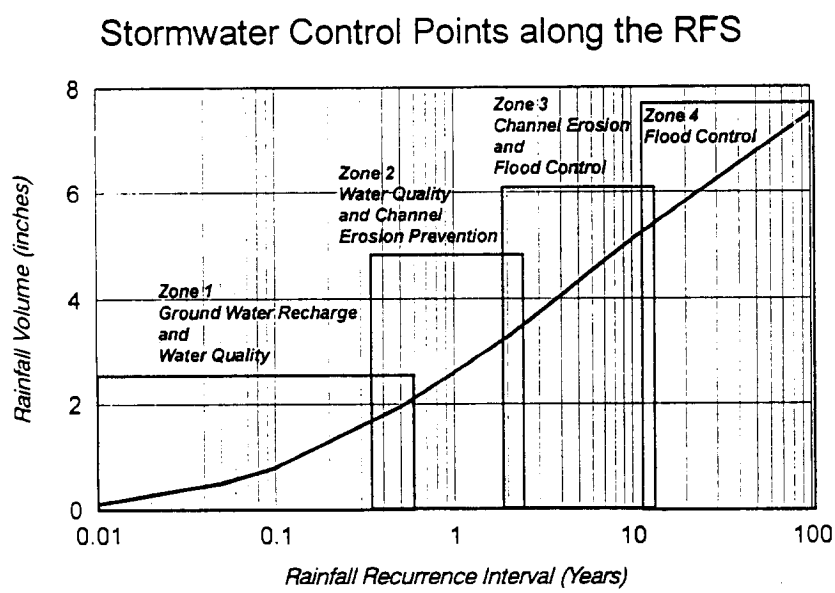
Larger rainfall events correspond reasonably well with SCS CN procedures.
Smaller rainfall events produce more runoff than is predicted by SCS CN procedures.
For strictly pervious surfaces, published CN's are much lower than observed CN's for small storm events. Therefore, less runoff is predicted from pervious areas during small storm events and SCS methodology incorrectly attributes more flow to impervious surfaces. This translates into inaccurate pollutant loading estimates from both pervious and impervious surfaces.
For impervious surfaces, the type of surface (i.e., rooftop, large paved surface, narrow street) has a significant impact on the amount of runoff for small storm events. The infiltration characteristics of these surfaces vary greatly. Remarkably, narrow streets can have a higher infiltration capability than some compacted urban pervious surfaces (such as ballfields).
Disconnecting impervious surfaces can significantly reduce the volume of runoff. The relative amount of reduction is a function of the pervious area flow path, the amount of impervious area draining to pervious areas, and the infiltration capacity of the pervious surfaces. Substantial reductions in runoff are observed for a wide range of land uses when impervious surfaces are disconnected and drained through permeable soils (SCS, Hydrologic Soil Groups (A and B). Reductions are only slight for relatively low density land uses when impervious surfaces are disconnected and drained through relatively impermeable soils (HSG's - C and D). Not surprisingly, disconnecting paved surfaces and rooftops for commercial areas does not result in significant reductions in runoff.

2.4 RAINFALL FREQUENCY SPECTRUM (RFS)

The effectiveness of any stormwater water quality treatment practice is a function of how much stormwater runoff is treated by the system and how much bypasses the practice. Since storms vary dramatically in magnitude, stormwater best management practices must be sized to capture a reasonable percentage of all runoff but bypass excessively large events. The rainfall frequency spectrum or RFS, which is defined as the distribution of all rainfall events, is a useful tool for establishing water quality treatment volume sizing criteria. This distribution is the cumulative volume from all storm events ranging from the smallest most frequent events in any given year to the largest most extreme events over a long duration, say, the 100 year frequency event.

The RFS consists of classes of frequencies often broken down by return interval, such as the two year storm return interval. Four principle classes are typically targeted for control by stormwater management practices. The two smallest, most frequent, classes are often referred to as water quality storms, where the control objectives are groundwater recharge, pollutant load reduction, and to some extent, control of channel erosion producing events. The two larger classes are typically referred to as quantity storms, where the control objectives are channel erosion control, overbank control, and flood control. Figure 2.5 illustrates a theoretical representation of these four classes.

FIGURE 2.5: FOUR CLASSES OF RAINFALL DISTRIBUTION



The distribution and magnitude of the RFS varies from region to region and to some extent, from year to year. Therefore, in order to establish a reasonable water quality treatment design volume for stormwater filtering practices it is necessary to define the RFS for the region of application. Within the Chesapeake Bay Watershed the average precipitation characteristics vary somewhat. This manual presents a sizing criteria based on an in-depth analysis conducted for the Washington, DC metropolitan area, compared with three other locations within the Bay and makes

recommendations for establishing the RFS for other locations within the Bay Watershed.

Schueler (1987 and 1992), conducted a detailed evaluation of 50 years of hourly rainfall data in the Washington D.C. area. The recorded precipitation data from Washington National Airport consisted of all storm events separated by at least 3 hours from the next event. The base data collected at National Airport included minor storm events which normally do not produce measurable runoff. These minor events make up approximately 10% of all annual rainfall, are usually less than 0.1 inches, and are therefore excluded from the RFS analysis.

Table 2.11 outlines the RFS for the Washington D.C. metropolitan area and illustrates that the vast majority of all annual runoff is produced from the small frequent storm events.

TABLE 2.11: RAINFALL FREQUENCY SPECTRUM WASHINGTON, DC AREA[a]
SOURCE: DESIGN OF STORMWATER WETLAND SYSTEMS (SCHUELER, 1992)

Percent of All Storm Events [b]	*Return Interval*	*Rainfall [c] Volume*
30	7 days	0.25
50	14 days	0.40
70	Monthly	0.75
85	Bi-monthly	1.05
90	Quarterly	1.25
95	Semi-annually	1.65
98	Annually	2.40
99	Two-year	2.90

a. 50 year analysis of hourly rainfall record at Washington National Airport, excluding all storms less than 0.10 inches that were separated by three consecutive hours from the next storm. These small storms seldom produce measurable stormwater runoff, yet are numerically the most common rainfall event.

b. Equal to or less than given rainfall volume

c. Watershed inches

2.5 THE 90% RULE-CUMULATIVE RAINFALL VOLUME FOR WATER QUALITY TREATMENT

A careful examination of Table 2.11 suggests that a BMP which is sized to capture and treat the three month storm frequency storm (or 1.25" rainfall) will effectively treat 90% of the annual average rainfall. While this is true, such a practice will also capture and at least partially treat the first 1.25" of larger rainfall events. Therefore treating the 1.25" rainfall will result in a capture efficiency of greater than 90%.

Given the economic considerations of capturing and storing a reasonably large water quality volume, and the realization that stormwater filters tend to lose efficiency as pollutant load input concentrations decrease (Bell, et. al, 1995), a smaller storm event was investigated to evaluate the effectiveness of an alternative treatment criteria. Many jurisdictions require storage of the first one-half inch of runoff from impervious surfaces. While this volume appears to have gained widespread acceptance, there has been little research on the cumulative pollutant load bypassing facilities sized on this principle. One notable exception, is a study conducted in Texas by Chang and his colleagues (1990), where the annual total solids load captured using the half-inch rule showed significant drop-off when imperviousness approached 70%.

To balance the desire to capture and treat as much cumulative rainfall as possible while avoiding an overly burdensome sizing criteria, additional rainfall data was evaluated throughout Chesapeake Bay watershed. In addition to Washington, DC, Three other locations were selected to evaluate longer term rainfall characteristics.

Daily precipitation data was analyzed for an 11 year period (January 1980 through December 1990) at four locations within the Chesapeake Bay Watershed. Norfolk VA, Washington, DC, Frederick MD, and Harrisburg, PA were selected as representative of the bay-wide watershed where new development activity is occurring. In addition locations are separated by 100 to 150 miles and represent a distribution from coastal to inland, and south to north.

The one-inch rainfall was evaluated to assess whether this value could be used to effectively capture 90% of the annual runoff. The average capture percentage using the 1.0" rainfall ranges from approximately 85% to 91% for the four locations. The analysis included the first one-inch of larger rainfall events which will be captured, but probably not completely treated. It is recognized that during these large events treatment conditions may be less than ideal. But it is safe to say that approximately 90% of the annual average rainfall events will be captured and treated using a **one-inch rainfall criteria**.

The results presented in Table 2.12 provide justification for using the 1.0" rainfall event for sizing stormwater filtering practices throughout the Chesapeake Bay Watershed. It must be emphasized that regional rainfall characteristics will differ from specific location to location. Additional rainfall frequency analysis is required for more complete reliance on this value. If a particular jurisdiction has the resources and long term data, a complete RFS should be conducted and the 90% rule applied to establish a local water quality precipitation value. In addition a longer data-set (say 50 years) will make some of the extreme rainfall events or drought periods less statistically significant and may have a minor effect on the capture value derived herein.

TABLE 2.12: COMPARISON OF PRECIPITATION DATA FOR FOUR LOCATIONS WITHIN THE CHESAPEAKE BAY WATERSHED 1980 - 1991 (DAILY ANALYSIS)

	Norfolk, VA	*Washington, DC*	*Harrisburg, PA*	*Frederick, MD*
Annual average precipitation	43.4 inches	37.9 inches	39.6 inches	37.0 inches
Annual average snowfall	7.7 inches	17.2 inches	31.3 inches	Not Obtained
Annual average # of precipitation days *	76 days	67 days	71 days	68 days
Annual average # of precipitation days more than 1.0"	10.5 days	9.5 days	9.5 days	7.7 Days
Annual average # of precipitation days less than 0.1"	39.0 days	45.4 days	55.1 days	Not Obtained
Percent of annual average rainfall ≤ 1.0" *	85.3%	91.4%	86.8%	89.9%
Percent of annual precipitation days ≤ 1.0" *	86.2%	85.9%	86.7%	88.6%

* adjusted to exclude rainfall events ≤ 0.1 (assumed to produce no runoff)

2.6 STORMWATER FILTERING SYSTEMS - SIZING CONSIDERATIONS

In general, stormwater filtering systems should be sized based on the **volume** of runoff to be filtered. All practices identified in this manual utilize the volume based sizing criteria, except for the grass channel practice, where a peak rate is utilized. It is necessary, however, to utilize a peak rate of discharge for sizing off-line flow diversion structures.

As presented earlier in this chapter, the target rainfall event for estimating the Water Quality Volume (WQV) for sizing all filtering devices is based on the **90% Rule** for capturing annual runoff volume. For the Mid-Atlantic region and much of the Chesapeake Bay Watershed, a rainfall value of **1.0 inches** is suggested.

Some jurisdictions may elect to use other sizing guidelines, such as the ½ inch rule (measured in watershed inches). This criteria may be acceptable for lower imperviousness but will have decreased pollutant capture efficiencies for a higher imperviousness and a lower capture percentage of the annual runoff volume. The individual practice sizing principles contained in this manual are applicable for alternative treatment volumes so a reliance on the 90% Rule is not mandatory. In addition, several filtering practices are ideally suited for retrofit applications where full storage is often constrained. Designers and regulators should recognize that the 90% Rule is targeted mainly at new construction and is based on maximizing pollutant load capture. Practices sized for smaller treatment volumes are certainly acceptable in many situations.

2.7 ESTIMATING WATER QUALITY VOLUME (WQV)

Two methods can be utilized to estimate the Water Quality Volume (WQV). Both rely on computing a volumetric runoff coefficient (R_v) and multiplying this by the rainfall volume to obtain a runoff volume in watershed inches.

The first method, or what we call the **Short Cut Method,** utilizes equation 2.1 to estimate the volumetric runoff coefficient R_v, (Schueler, 1987). It is recommended that the Short Cut Method be utilized where the site consists of predominately one type of land surface or for quick calculations to obtain a reasonably accurate estimate of treatment volume.

$$R_v = 0.05 + 0.009(I)$$ **Equation 2.1**

where I = site percent impervious

Therefore, the required treatment volume for a site will be equal to:

$$WQV = P * R_v \qquad \textbf{Equation 2.2}$$

P = rainfall, in inches
and WQV = Water Quality Volume, in watershed inches

EXAMPLE CALCULATION

Assume a 3.0 acre shopping center which is 87% impervious, for a 1.0 inch rainfall event.

$R_v = 0.05 + 0.009(87\%)$
$R_v = 0.83$

for P = 1.0 inches
WQV = (1.0")(.83) = .83 watershed inches
WQV = .83"(1/12 "/ft)(3.0 ac)(43,560 ft^2/ac) = 9,039 ft^3

The second method, or **Small Storm Hydrology Method** utilizes the work done by Pitt and others, to compute a volumetric runoff coefficient (R_v) based on the specific characteristics of the pervious and impervious surfaces of the drainage catchment. This method presents a relatively simple relationship between rainfall amount, land surface, and runoff volume. The R_vs used to compute the volume of runoff are identified in Table 2.13. The small storm hydrology model involves the following:

- For a given rainfall depth, the runoff coefficients for land surfaces present on the subject site are selected.
- A weighted runoff coefficient for the entire site is computed.
- If a portion of the site has disconnected impervious surfaces, reduction factors are applied to R_v. The reduction factors (from Table 2.14) are multiplied by the computed R_v for connected impervious areas to obtain the corrected value.
- For the given rainfall, the runoff volume (in watershed inches) is computed. WQV is equal to the rainfall times the R_v (same as equation 2.2 above).

TABLE 2.13: VOLUMETRIC COEFFICIENTS FOR URBAN RUNOFF (DIRECTLY CONNECTED IMPERVIOUS AREAS, ADAPTED FROM PITT, 1994)

Rainfall (inches)	Flat roofs and large unpaved parking lots	Pitched roofs and large impervious areas (large parking lots)	Small impervious areas and narrow streets	Sandy soils HSG-A	Silty soils HSG-B	Clayey soils HSG-C & D
0.75	.82	.97	.66	.02	.11	.20
1.00	.84	.97	.70	.02	.11	.21
1.25	.86	.98	.74	.03	.13	.22
1.50	.88	.99	.77	.05	.15	.24

TABLE 2.14: REDUCTION FACTORS TO VOLUMETRIC RUNOFF COEFFICIENTS FOR DISCONNECTED IMPERVIOUS SURFACES (ADAPTED FROM PITT, 1994)

Rainfall (inches)	Strip commercial and shopping center	Medium to high density residential with paved alleys	Medium to high density residential without alleys	Low density residential
0.75	.99	.27	.21	.20
1.00	.99	.38	.22	.21
1.25	.99	.48	.22	.22
1.50	.99	.59	.24	.24

In order to use the reduction factors for disconnected impervious surfaces, as general guidance, the impervious area above the pervious surface area should be less than one-half of the pervious surface and the flowpath through the pervious area should be at least twice the impervious surface flowpath.

The Small Storm Hydrology method has the advantage of evaluating the precise elements of a particular site and should be utilized for most design applications to estimate accurate runoff volumes. The method requires somewhat more effort to identify the

specific land surface area ratios and additional effort is needed to assess the disconnections of impervious areas. The method rewards site designs which utilize disconnections of impervious surfaces by lowering the computed R_v and the required WQV.

EXAMPLE CALCULATION

Assume a 3.0 acre small shopping center having a 1.0 acre flat roof, 1.6 acres of parking and a 0.4 acre open space (sandy soil), for a 1.0 inch rainfall event and no disconnection of impervious surfaces. The weighted volumetric runoff coefficient is:

flat roof:	1.0 acre x .84 = 0.84
parking:	1.6 acres x .97 = 1.55
open space:	0.4 acre x .02 = 0.01
total:	3.0 acres = 2.40

weighted volumetric runoff coefficient R_v = 2.40/3.0 = .80

for P = 1.0 inches
Water Quality Volume (WQV) = (1.0")(.80) = .80 watershed inches
= (.80") (1 ft/12") (3.0 ac) (43,560 ft^2/ac)
= 8,712 ft^3

2.8 ESTIMATING PEAK DISCHARGE FOR THE WATER QUALITY STORM (Q_P)

The peak rate of discharge is needed for the sizing of off-line diversion structures and to design grass channels. As discussed earlier in this chapter, conventional SCS methods underestimate the volume and rate of runoff for rainfall events less than 2". This discrepancy in estimating runoff and discharge rates can lead to situations where a significant amount of runoff by-passes the filtering treatment practice due to an inadequately sized diversion structure or leads to the design of undersized grass channels.

The following procedure can be used to estimate peak discharges for small storm events. It relies on the volume of runoff computed using the Small Storm Hydrology Method and utilizes SCS, TR-55 Graphical Peak Discharge Method.

- Using the water quality volume (WQV), computed using the methods previously presented, a corresponding Curve Number (CN) is computed utilizing equation 2.3.

$$CN = 1000/[10 + 5P + 10Q - 10(Q^2 + 1.25\,QP)^{1/2}]$$ **Equation 2.3**

where P = rainfall, in inches (use 1.0" for the Water Quality Storm)
and Q = runoff volume, in inches (equal to WQV)

Note: Equation 2.3 above, is derived from the SCS Runoff Curve Number method described in detail in NEH-4, Hydrology (SCS 1985) and SCS TR-55 Chapter 2: Estimating Runoff. The CN can also be obtained graphically (also from TR-55).

- Once a CN is computed, the time of concentration (t_c) is computed (based on the methods identified in TR-55, Chapter 3: "Time of concentration and travel time"). The t_c for small sites is often small based on relatively short flow paths; however, a minimum value of 0.1 hours should be used.

- Using the computed CN, t_c and drainage area (A), in acres; the peak discharge (Q_p) for the Water Quality Storm is computed (based on the procedures identified in TR-55, Chapter 4: "Graphical Peak Discharge Method"). For the Chesapeake Bay Watershed use Rainfall distribution type II.

  - Read initial abstraction (I_a), compute I_a/P
  - Read the unit peak discharge (q_u) from Exhibit 4-II for appropriate t_c
  - Using the water quality volume (WQV), compute the peak discharge (Q_p)

$$Q_p = q_u * A * WQV$$ **Equation 2.4**

where Q_p = the peak discharge, in cfs
q_u = the unit peak discharge, in cfs/mi²/inch
A = drainage area, in square miles
and WQV = Water Quality Volume, in watershed inches

EXAMPLE CALCULATION

Using the previous example:

where WQV = .80"

$CN = 1000/[10+5*1.0"+10*.80"-10((0.80")^2+1.25*.80"*1.0")^{1/2}]$

CN = 98

assume t_c = 10 minutes = .17 hours

I_a = 0.041 for CN = 98, I_a/P = 0.041/1.25" = .03

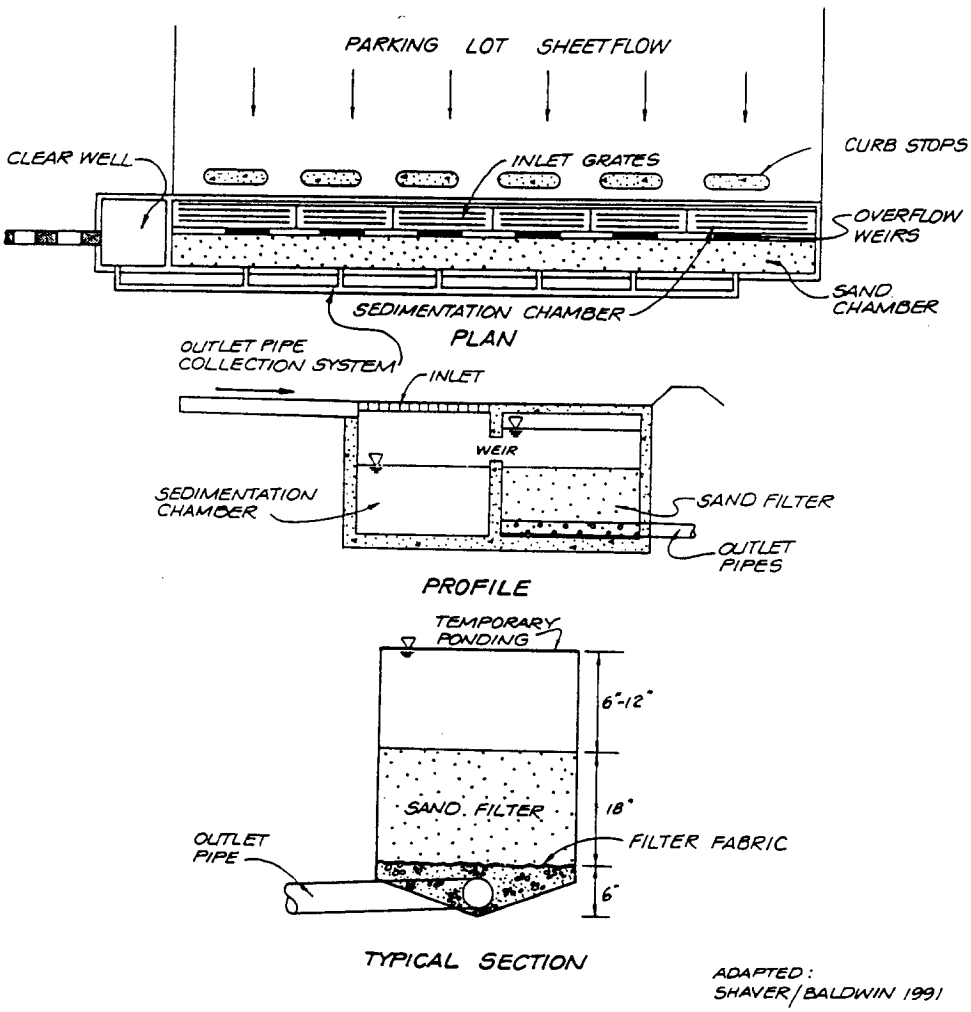
read q_u = 950 csm/in (TR-55 Exhibit 4-II)

A = 3.0 acres/640ac/mi² = .0047mi²

Q_p = 950 csm/in * .0047mi² * .80" =3.6 cfs

For computing runoff volume and peak rate for storms larger than the Water Quality Storm (i.e., 2, 10 and 100 year storms), use the published CN's from TR-55 and follow the prescribed procedure in TR-55.

In some cases the Rational Formula may be used to compute peak discharges associated with the Water Quality Storm. The designer must have available reliable intensity, duration, frequency (IDF) tables or curves for the storm and region of interest. This information may not be available for many locations and therefore the TR-55 method described above is recommended.



PERIMETER SAND FILTER

Chapter 3

Selecting the Right Filter for a Site

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CHAPTER 3
SELECTING THE RIGHT FILTER FOR A SITE

This chapter presents guidance for selecting the most appropriate stormwater filter for a particular development site. This information has been condensed in a series of tables that help designers and municipal officials select the most effective stormwater filter for their situation. In addition, the chapter compares stormwater filters against other stormwater practices that also could be applied at the site (e.g., ponds, wetlands, and infiltration systems) Again, a series of tables examine the comparative pollutant removal, feasibility criteria and environmental benefits of the four groups of stormwater BMPs.

3.1 SELECTING THE BEST STORMWATER FILTER DESIGN

Given that there are at least eleven different stormwater filter designs that can be used, what is the best option for each site? Quite simply, three factors need to be considered when making the choice. First, is the filter appropriate for the type of development being considered? Second, do site conditions such as space consumption, available head, cost or maintenance consideration favor the use of the design? Third, how effective is the stormwater filter design in removing the key pollutants of concern? Usually, by the time all three questions are answered, the filtering options are narrowed down to one or two design options. The engineer can then compare the design criteria for the remaining options and select one based on cost and effectiveness.

3.1A MOST APPROPRIATE DEVELOPMENT CONDITIONS FOR STORMWATER FILTERS

Although as a group, stormwater filters can be applied to a diverse range of development conditions, individual designs are limited to a much narrower range. These common development situations include ultra-urban sites, parking lots, road and streets, small residential subdivisions and backyard/rooftop drainage. Table 3.1 is a matrix that illustrates the most economical and feasible filtering designs for each of these five broad categories of development, as well as those that are not applicable.

For example, in *ultra-urban or retrofit* settings where space is at a premium, the underground sand filter is often the most ideal filtering design, although surface, perimeter, and pocket sand filters, as well as gravel filters may also be considered. In most cases, the space requirements of grass channels, swales and filter strips are so great that they can be eliminated from consideration.

TABLE 3.1: MOST APPROPRIATE FILTER OPTION FOR DIFFERENT LAND USES

Filtering System	*Ultra-Urban or Retrofit*	*Parking Lots*	*Roads and Highways*	*Residential*	*Pervious*	*Rooftops*
Surface Sand	Yes	Ideal	Depends	Depends	No	Yes
Underground Sand	Ideal	Yes	No	Depends	No	Yes
Perimeter Sand	Yes	Ideal	Depends	Depends	No	Yes
Organic	Depends	Yes	No	Depends	Depends	Yes
Pocket Sand	Yes	Yes	Depends	Yes	No	Yes
Grass Channel	No	No	Depends	Depends	Yes	Depends
Dry Swale	No	Depends	Ideal	Ideal	Yes	Depends
Wet Swale	No	Depends	No	No	Depends	Depends
Bioretention	Depends	Ideal	Yes	Yes	Yes	Yes
Filter Strip	No	Depends	Yes	Yes	Ideal	Yes
Gravel Filter	Yes	Yes	No	No	Depends	Depends

Notes:
Ideal: **Physically and economically the best alternative for a site**
Depends: **May be suitable under certain conditions (space, soils, water table)**
Yes: **Generally suitable for most development projects within category**
No: **Seldom or never suitable**

Three filtering systems are considered ideal to treat the quality of stormwater runoff from *parking lots*- the surface and perimeter sand filter and bioretention areas.

While several other filtering options are also feasible at parking lots, these three appear to be the most effective and economical designs. (It should be noted that filter strips may be an excellent choice on smaller parking lots adjacent to stream buffers or open space).

The linear nature of *streets and highways* make the grass channel an excellent

choice for this kind of development. A grass channel usually fits within the right of way, and is a relatively simple adaption of the drainage channel that is usually provided anyway. If a greater degree of pollutant removal is desired, a dry or wet swale might be contemplated, or an off-line bioretention area. Generally, sand, gravel or organic filter designs are not practical for roads and highways.

In *residential subdivisions*, the preferred designs are the dry swale, bioretention, and the pocket sand filter. When designed properly, each design blends into the landscape and has relatively low maintenance requirements. Sand filters and gravel filters may not be a very practical option in most small subdivisions because of their high maintenance requirements and appearance. Homeowners also have shown little preference for wet swales that are hard to mow and may cause nuisance problems. Once a residential subdivision exceeds a density of 4 dwelling units per acre, few filtering designs of any kind are practical (due to drainage area limitations). Larger systems, such as ponds and wetlands, are probably a better choice.

The filter strip is considered the most ideal option for dealing with *rooftop* runoff and lawns in residential areas, although bioretention is also a practical solution. These two designs can effectively treat stormwater quality at low cost with only minor changes in site grading, assuming land is available. Rooftop runoff from *industrial and commercial* sites may be effectively treated within sand and organic filters, or gravel filters. A higher level of treatment and a concern for groundwater protection make these three options the best choice.

3.1B KEY FEASIBILITY CRITERIA FOR STORMWATER FILTERS

What site conditions make a stormwater filtering design infeasible at a site? While stormwater filters are subject to fewer site constraints than other types of BMP systems, there are a few key factors that should be screened in the selection process including space consumption, minimum head, maintenance burden, cost and soil conditions. These feasibility factors are compared in a matrix format for each of the eleven stormwater filters in Table 3.2.

Space consumption is probably the most critical factor affecting the selection of a stormwater filter design. To compare the practices on the same basis, space consumption is expressed as a practice surface area as a percentage of the contributing impervious acreage. As can be seen, filtering designs run the full range of space consumption with sand filters at the low end of the range (2 to 3%), grass channels and bioretention in the mid-range (about 5%), swales (10 to 20%), and filter strips (100%) at the high end. Thus, if available space is tight, a practice could very well be eliminated.

Most filtering designs require a *minimum vertical distance* (or fall) be available from the inflow to the filter to its outflow point. This distance, known as head, is necessary to drive water through the entire filtering system by gravity. In nearly every filter design, at least two feet of head are needed. Most sand filter designs require about five feet. If a site has very low relief, a filter design may not be practical for the site.

A third key feasibility factor is the *cost* of constructing the filtering system, and again, the eleven designs exhibit a wide range. The most expensive designs, based on the cost per impervious acre treated, are the underground sand, organic sand, perimeter sand and gravel filters. The pocket sand filter and dry swale are in the mid-cost range, whereas bioretention, wet swales, filter strips and grass channels are very attractive options from a cost standpoint. It should be noted that the construction cost does not include the price of land. If land costs are significant, the rank-order changes dramatically.

TABLE 3.2: COMPARISON OF STORMWATER FILTERING SYSTEM OPTIONS

Filtering System	*Space Consumed**	*Minimum Head*	*Maintenance Burden*	*Cost $$*	*Other Notes*
Surface Sand	2-3%	5-8 feet	Annual	Moderate	Large Sediment
Underground Sand Filter	None	5-6 feet	Semi-Annual Cleanouts	High	OSHA Confined Space
Perimeter Sand Filter	2%	2-3 feet	Annual Cleanouts	Moderate	Located Outside Curbstops or in Travelway
Organic Sand Filter	2-3%	5-8 feet	Annual Cleanouts	High	Filter Replacement
Pocket Sand Filter	2%	5 feet	Cleanouts	Moderate	Level Spreader
Grass Channel	6.5%	2 feet	Mowing	Low	Checkdams
Dry Swale	10-20%	2-6 feet	Mowing	Moderate	Prepared Soil
Wet Swale	10-20%	2-6 feet	Wetland	Low	Standing Water
Bioretention	5.0%	4 feet	Landscaped	Low	Plant Selection
Filter Strip	100%	2 feet	Edge Scraping	Low	Contributing Flow Length 75' or 150' max
Gravel Filter	3-5%	2-4 feet	Wetland Cleanouts	High	Standing Water

*Approximate % of total site impervious area draining to practice

Table 3.2 also compares how each filtering design rates with respect to *maintenance burden* and other important feasibility factors.

3.1c COMPARATIVE POLLUTANT REMOVAL CAPABILITY

How effective are the filtering designs at removing the key pollutants of concern in a watershed? As part of the preparation of this manual, some thirty published and unpublished monitoring studies were consulted on the pollutant removal performance of stormwater filtering systems. Estimated average removal rates for each of the eleven stormwater filter designs are indicated in Table 3.3. The matrix also shows the number of actual performance monitoring studies that were available to assess a given design. Three filtering designs (underground sand filters, pocket sand filters and bioretention) have yet to be monitored, and their potential performance is inferred from monitoring of similar designs, infiltration rates, modeling and other analysis provided in Chapter 4.

Despite their many differences in design, stormwater filters have some similarities with respect to performance. For example, all typically report removal rates of *suspended sediment* in excess of 80%. Although monitoring data for *hydrocarbons* is more limited, removal rates typically ranged from 65% to 90%.

Some differences were seen in the comparative ability to remove *total phosphorus*. The best performers were the surface and perimeter sand filter, dry swale and gravel filter, all of which showed at least a 50% removal. Grass channels, wet swales, filter strips and possibly organic sand filters were less reliable, at 10 to 40% average removal.

Stormwater filtering systems exhibit only a modest capacity to remove *total nitrogen*. only one design was found to remove more than 50% of total nitrogen (gravel filter), and most ranged from 30 to 45%. The bulk of the observed removal was for organic forms of nitrogen; eight of eleven filtering designs had zero or even negative removal rates for soluble nitrate-nitrogen. The latter phenomena reflects the fact that while nitrification is prevalent in the mainly aerobic environment of most filter beds, denitrification is limited (leading to buildup of nitrate in the effluent). Only the gravel filter, dry swale, and wet swale showed a capability to remove nitrate.

While all filtering designs showed at least moderate capacity to remove *trace metals* such as copper, lead, and zinc, most of the removed metals were already attached to particles. Designs that showed promise in removing dissolved metals include the organic sand filter, gravel filter and dry swale.

TABLE 3.3: ESTIMATED POLLUTANT REMOVAL CAPABILITY OF DIFFERENT STORMWATER FILTER SYSTEMS (AVERAGES OF REPORTED MONITORING DATA)

Filtering System	*Monitoring Data?*	*TSS*	*TP*	*TN*	*NO^3*	*Other Pollutants/Comments*
Surface Sand Filter	Yes, 6	85%	55%	35%	Neg	Bacteria: 40-80% Metals: 35-90%
Underground Sand Filter	No Data	Presumed to Comparable to Surface Sand Filter				
Perimeter Sand Filter	Yes, 3	80%	65%	45%	Neg	Hydrocarbons: 80%
Organic Sand Filter	Yes, 1	95%	40%	35%	Neg	Hydrocarbons: 90% Sol. P Negatives Metals: 85%+
Pocket Sand Filter	No Data	Presumed to be Comparable to Surface Sand Filter				
Drainage Channel	Yes, 10	30%	10%	Zero	Zero	Bacteria: Negative
Grass Channel = biofilter	Yes, 1	65%	25%	15%	Neg	Hydrocarbons: 65% Metals: 20-50% Bacteria: Negative
Dry Swale	Yes, 3	90%	65%	50%	80%	Metals: 80-90%
Wet Swale	Yes, 2	80%	20%	40%	50%	Metals: 40-70%
Bioretention	No Data	Presumed to be Comparable to Dry Swale				
Filter Strip	Yes, 1	70%	10%	30%	Zero	Metals: 40-50%
Gravel Filter	Yes, 2	80%	80%	65%	75%	Hydrocarbons: 85% Metals: 50-75%

Control of *fecal coliform bacteria* is important in shellfish areas, beaches and drinking water supplies. The filter designs that showed the best ability to remove bacteria included surface sand filters and gravel filters; drainage channels and grass channels had no effect on bacterial levels, and the remaining practices have yet to be monitored for this important parameter.

It should be noted that pollutant removal rates and mechanisms rely on processes in a generally aerobic environment, as opposed to anaerobic environment. Filters which go anaerobic tend to release previously captured phosphorous as iron phosphates break down.

3.1D COMPARATIVE DESIGN CRITERIA

The sizing criteria for each of the eleven filtering designs are summarized in Table 3.4. Each type of filter design is compared based on the sizing criteria for each of its four standard design components:

- the quantity and method used for flow regulation
- the quantity and method used for pretreatment
- the depth and nature of the filter media and the area of the filter bed, expressed as the percentage of contributing impervious area
- the quantity and method used for overflow

TABLE 3.4: COMPARATIVE DESIGN CRITERIA FOR STORMWATER FILTERS

Filter Type	Design Criteria by System Component			
	Flow Regulation Quantity and Method	***Pretreatment Method and Quantity***	***Filter Bed and Media***	***Overflow***
Surface Sand	WQV by volume, within facility Entire treatment system must hold at least ¾ of WQV	Dry Sedimentation for 24 hours, approx. ½ of minimum treatment volume (¾ of WQV)	Size based on Darcy's law, coeff. of permeability (k) = 3.5 ft/day, approx. 1% of impervious drainage area. 18"-24" thick sand	Gravel/pipe underdrain system, overflow weir sized to pass ⅓ of WQV from filter bed.
Underground Sand	WQV by rate, within drainage system. Entire treatment system must hold at least ¾ of WQV	Wet Retention, approx. ⅔ of min. treatment volume (¾ of WQV)	Size based on Darcy's law, coeff. of permeability (k) = 3.5 ft/day, approx. 1% of impervious drainage area. 18"-24" thick sand	Gravel/pipe underdrain system, overflow weir sized to pass ⅔ of WQV from filter bed.
Perimeter Sand	WQV by volume, within facility. Entire treatment system must hold at least ¾ of WQV	Wet Retention, approx. ⅔ of min. treatment volume (¾ of WQV)	Size based on Darcy's law, coeff. of permeability (k) = 3.5 ft/day, approx. 1% of impervious drainage area. 18" thick sand	Gravel/pipe underdrain system, overflow weir sized to pass 100% of WQV from filter bed.
Pocket Sand	WQV by rate, within drainage system. Entire treatment system must hold at least ¾ of WQV	Vegetated strip and plunge pool, no minimum volume	Size based on Darcy's law, coeff. of permeability (k) = 3.5 ft/day, approx. 1% of impervious drainage area. 18"-24" thick sand	Gravel/pipe underdrain system, overflow weir sized to pass 100% of WQV from filter bed.
Organic Media	WQV by volume, within facility.* Entire treatment system must hold at least ¾ of WQV	Dry Sedimentation for 24 hours, approx. ½ of minimum treatment volume (¾ of WQV)*	Size based on Darcy's law, coeff. of permeability (k) = 4.3 ft/day (peat), 8.7 ft/day (compost), less than 1% of impervious drainage area. 24" thick peat/sand profile consisting of 12" thick peat, 4" thick peat/sand mix, and 8" sand. or 18" thick compost	Gravel/pipe underdrain system, overflow weir sized to pass ⅓ of WQV from filter bed.*
Bioretention	On-line, up to WQV, then overflow	Pea gravel diaphragm and 10' min. vegetated filter strip, no min. volume.	Size based on Darcy's law, coeff. of permeability (k) = 0.5 ft/day, approx. 5.0 % of impervious drainage area. 4' thick planting soil bed, and overlying mulch layer.	Gravel/pipe underdrain system, overflow drainage catchbasin sized to pass design peak discharge (e.g., Q_{10})

Filter Type	Design Criteria by System Component			
	Flow Regulation Quantity and Method	***Pretreatment Method and Quantity***	***Filter Bed and Media***	***Overflow***
Grass Channel	On-line, rate based on Q_p from WQV, velocity ≤1.5 fps	Pea gravel diaphragm and vegetated filter strip, forebay at inflow, no min. volume.	Rate based design, minimum residence time = 10 min. Depending on slope, treatment area approx. = 6.5% of impervious drainage area. Grass surface/soil interface	On-line flow, sized to treat WQV with velocity ≤ 1.5 fps, 2 year non-erosive velocities (≤ 4.0 to 5.0 fps), adequate capacity for 10 year storm with 6" freeboard.
Dry Swale	On-line, volume based on WQV	Pea gravel diaphragm and vegetated filter strip, forebay at inflow, no min. volume.	Volume based design to retain WQV. Depending on slope and depth, treatment area approx. = 16% of impervious drainage area. 30" thick planting soil bed, consisting of 50% soil/50% sand mix.	On-line flow, sized to treat WQV, 2 year non-erosive velocities (≤ 4.0 to 5.0 fps), adequate capacity for 10 year storm with 6" freeboard.
Wet Swale	On-line, volume based on WQV	Pea gravel diaphragm and vegetated filter strip, forebay at inflow, no min. volume.	Volume based design to retain WQV. Depending on slope and depth, treatment area approx. = 16% of impervious drainage area. Grass/wetland vegetation surface/soil interface	On-line flow, sized to treat WQV, 2 year non-erosive velocities (≤ 4.0 to 5.0 fps), adequate capacity for 10 year storm with 6" freeboard.
Filter Strip	On-line volume based on WQV	Pea gravel diaphragm, no min. volume	Volume based design to retain WQV. Depending on slope and depth, treatment area approx. = 100% of impervious drainage area. Grass surface/soil interface	On-line flow, sized to treat WQV, all other flows overflow berm
Gravel Wetland Biofilter	WQV by volume, within facility	Wet Retention, approx. ⅔ of min. treatment volume (¾ of WQV)	24" recycled concrete or bank-run gravel	

* assumes same design variation as surface sand filter

3.2 COMPARISON TO OTHER BMPS

Stormwater filtering systems are just one of four groups of stormwater best management practices that can be used to treat the quality of stormwater runoff. The other three groups include:

- stormwater ponds
- stormwater wetlands, and
- stormwater infiltration systems.

Since these practices can also be applied at development sites, it is helpful to compare their advantages and disadvantages with respect to stormwater filtering systems. Again, a series of matrices are used to make generalized comparisons based on physical feasibility, pollutant and environmental restrictions.

3.2A COMPARATIVE FEASIBILITY

A quick glance at Table 3.5 suggests that stormwater filtering systems are subject to far fewer physical restrictions than the other three groups of practices. For example, stormwater filters are feasible option on smaller drainage areas, have no soil restrictions, and have relatively modest head and space requirements. Ponds and wetlands, by contrast, typically require larger drainage areas (ten acres or more) and infiltration practices are often severely restricted by soil conditions.

On the other hand, stormwater filters are generally not a cost-effective option beyond a five acre drainage area, and need frequent cleanouts to maintain the performance of the filter bed (1 to 3 years). In addition, both filtering and infiltration systems are not commonly used to provide stormwater quantity controls for the larger design storms (2, 10 and 100 year events). These must be treated in a downstream detention or retention facility.

TABLE 3.5: FEASIBILITY CRITERIA FOR DIFFERENT STORMWATER BMP OPTIONS

Feasibility Criteria	*Pond Systems*	*Wetland Systems*	*Infiltration Systems*	*Filter Systems*
Soils	Most Soils	Most Soils	Need Infiltration Rate .5"/Yr or	All Soils
Drainage Area	10 Acre Min.	10 Acre Min.	2-5 Acre Max.	2-5 Acre Recommended
Head	3-6 Feet	1-6 Feet	2-4 Feet	1-8 Feet
Space	2-3% of Site	3-5% of Site	2-3% of Site	2-7% of Site
Cost/Acre	Low	Moderate	High	Moderate-High
Water Table	No Restrictions	No Restrictions	4 Feet Below	2 Feet Below Filter Bottom
Cleanout	2-10 Years	2-5 Years	1-2 Years	1-3 Years
SWM Mgmt.	Yes	Yes	No	No
Longevity	20-50 Years	20-50 Years	1-5 Years	5-20 Years?

3.2B COMPARATIVE POLLUTANT REMOVAL

The matrix in Table 3.6 compares the performance of stormwater filtering systems as a group with ponds, wetlands and infiltration. It is worth noting that this comparison is necessarily general, and several designs within each group may have better or worse performance than indicated in Table 3.6. (For more discussion, consult Section 4.4b in Chapter 4). A few of the general trends are outlined below.

All four practices generally display an excellent ability to remove *suspended sediment,* with filtering systems performing slightly better than the other three. Although filtering systems can perform as well as the other groups with respect to total phosphorus removal, they do show greater variability. Filtering systems generally have only a fair ability to remove nitrogen. Of particular concern, many filtering systems show a low ability to remove soluble nutrients that are of greatest concern in the Chesapeake Bay. Their tendency to export or leach nitrate and soluble phosphorus are well documented in Chapter 4, and few stormwater filters have an effective method for biological uptake by algae or wetland plants.

On the positive side, several filtering systems are documented to have a high to excellent ability to remove bacteria, metals, and hydrocarbons, which is equal or greater to that of ponds, wetlands and infiltration (although there are some data gaps).

In summary, the overall pollutant removal capability of stormwater filtering systems is on par with that for the other three systems, with higher removal for some pollutants, and lower removal for others.

TABLE 3.6:COMPARATIVE POLLUTANT REMOVAL CAPABILITY OF FOUR BMP OPTIONS

Pollutant	*Pond Systems*	*Wetland Systems*	*Infiltration System*	*Filter Systems*
Sediment	Excellent	Excellent	Excellent	Excellent
Phosphorus	High	High	Excellent	Fair-High
Nitrogen	Fair	Fair	High	Fair
Soluble Nutrients	High	Fair	High	Low
Bacteria	Low-High	?	?	Low-Fair
Hydrocarbons	High	High	?	Excellent
Trace Metals	Fair-	Fair-Excellent	High	Fair-Excellent
Key:	**Lo = 0-25% removal** **High = 51-75%**		**Fair = 26-50%** **Excellent = 76% or higher**	

3.2c ENVIRONMENTAL RESTRICTIONS AND BENEFITS

A key factor in the selection and permitting of urban BMPs are environmental restrictions and benefits. Table 3.7 compares stormwater filters with ponds, wetlands and infiltration systems in regard to ten common environmental restrictions. The matrix suggests that while there is little environmental risk associated with most stormwater filtering systems, they also confer few if any community benefits. For example, stormwater filters as a group generally pose little

risk of stream warming, groundwater contamination, wetland impairment and safety hazards. Each of the other practices is subject to one or more of these environmental constraints, which may make it difficult to obtain approval or permits in some subwatersheds. On the other hand, with few exceptions, most stormwater filter designs confer few of the environmental benefits that some of the other practices can provide, such as streambank protection, habitat creation and groundwater recharge. In addition, most stormwater filters are neutral with respect to community benefits (such as flood control, landscaping or increase in property value).

TABLE 3.7: ENVIRONMENTAL BENEFITS AND DRAWBACKS OF BMP OPTIONS

Selection Factor	*Pond Systems*	*Wetland Systems*	*Infiltration System*	*Filter Systems*
Groundwater Quality	Low Risk	Low Risk	Moderate Risk	No Risk
Groundwater Recharge	Moderate Benefit	Low Benefit	High Benefit	No Benefit (a)
Temperature	High Risk	High Risk	No Risk	Low Risk
Wetlands	High Risk	Moderate Risk	No Risk	Low Risk
Safety	High Risk	Low Risk	No Risk	No Risk
Habitat	Moderate Benefit	High Benefit	No Benefit	No Benefit
Flood Control	High Benefit	High Benefit	No Benefit (b)	No Benefit (b)
Streambank Protection	Moderate Benefit	Moderate Benefit	Low Benefit	Low Benefit (b)
Property Value	High Premium	Moderate Premium	No Premium	Unknown
Landscaping	High Benefit	High Benefit	No Benefit	Low Benefit

Notes: (a) Assumes filtering system has underdrain system
(b) Most do not control channel stability design storm events

3.3 SUMMARY

Stormwater filters are most applicable at small development sites, and can generally provide reliable rates of pollutant removal if design improvements are made and regular maintenance is performed. Stormwater filters appear to have particular utility in treating runoff from urban "hotspot" source areas such as commercial parking lots, vehicle service centers, and industrial sites, as well as problematic street and highway sites when other BMPs are not feasible. They are also an appealing alternative to other practices in environmentally-restricted watersheds. Lastly, it should be kept in mind that stormwater filters only treat the quality of runoff, and that a downstream detention or retention facility will be needed for stormwater management and flood control purposes.

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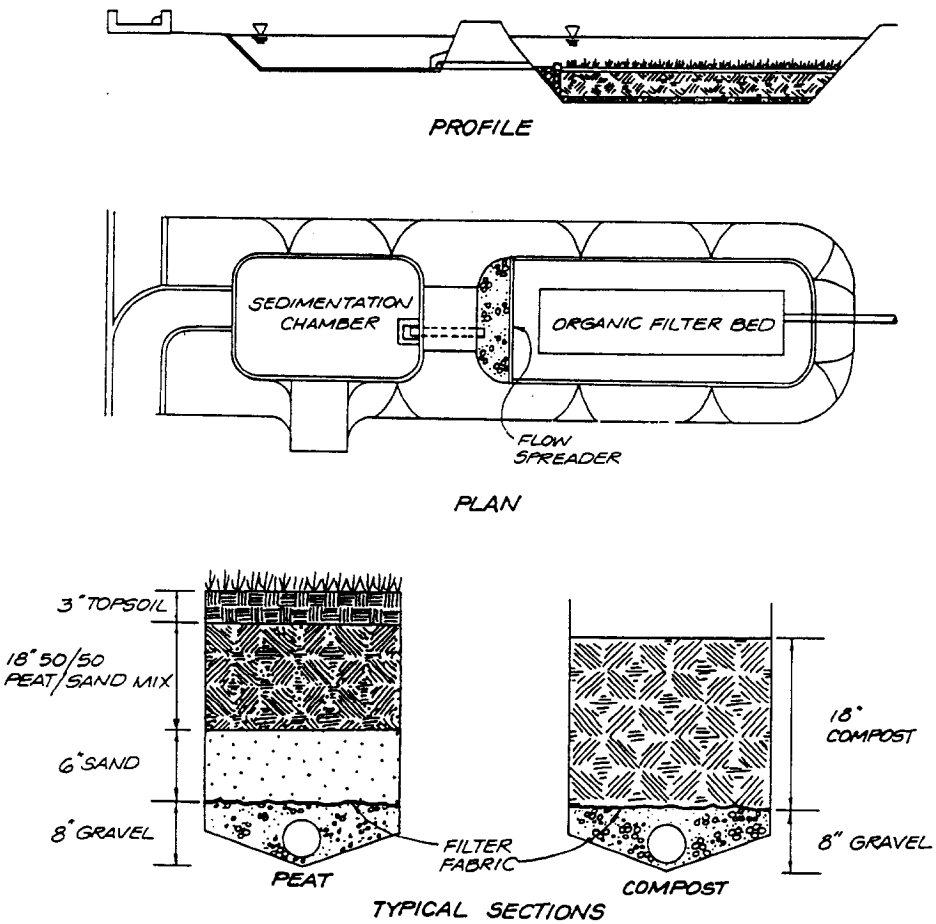

ORGANIC FILTER

CHAPTER 4
POLLUTANT REMOVAL MECHANISMS

This chapter explores the pollutant removal capability of five kinds of stormwater filtering systems, based on an analysis of published performance monitoring studies, engineering theory and basic research conducted around the country. The five groups of stormwater filtering systems that are considered include:

- sand and organic filters
- open channels
- filters strips
- bioretention
- vegetated submerged bed wetlands

The chapter describes the primary pollutant removal pathways associated with each group of practices, reviews reported pollutant removal performance monitoring principles and compares these rates to other stormwater BMP technologies. The discussion culminates in a series of key design principles to enhance pollutant removal performance that are incorporated into the design criteria in succeeding chapters.

4.1 POLLUTANT REMOVAL PATHWAYS

This section reviews the major pollutant removal pathways that occur within stormwater filtering systems. An understanding of these pathways is useful to interpret performance monitoring data, and to design more effective filtering practices. Each filtering practice utilizes a different combination of pollutant removal pathways which are compared in Table 4.1. The six primary removal pathways include:

4.1A PATHWAY NO. 1: SEDIMENTATION

Most urban BMPs rely heavily on gravitational settling as a primary pollutant removal pathway, and filtering systems are no exception. There are upper limits, however, to the amount of pollutant removal that can be achieved in this pathway. This is evident in the data of Stanley (1994) and Grizzard et al. (1983). In Stanley's recent study of an extended detention pond in North Carolina, modest to high removal of particulate pollutants that are prone to settling was observed, although removal of soluble pollutants was negligible (Table 4.2). Grizzard's settling column experiments demonstrate a similar behavior (Figure 4.1), with most removal

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occurring in the first six to twelve hours.

Most filtering systems incorporate a sedimentation chamber to settle out pollutants before runoff reaches the filter bed. The importance of settling as a pollutant removal pathway increases as more time and volume is provided for settling.

TABLE 4.1
COMPARISON OF POLLUTANT REMOVAL PATHWAYS IN STORMWATER FILTERS

Pollutant Removal	*Sand Filters*	*Organic Filters*	*Open Channel (a)*	*Filter Strip*	*Bio-retention*	*Gravel Filter*
Sedimentation	M, in pretreatment cell		M, surface cells	M, at grass strip interface	M, surface ponding	M, in pre-treatment cell
Straining	M, in filter media		L, surface soil	L, surface soil	H, sand, mulch, grass, soil	L, large pore spaces
Adsorption	by organics on filter surface	peat or sand	soil, thatch	soil, thatch	soil, mulch	biofilms on rocks
Microbe Action	On filter surface		L, soil	L, soil	M, soil/mulch	biofilms on rocks
Plant Uptake	None, unless grass cover crop used		L, by grass mowing	M, by grass or forest	M, by trees/ shrubs	H, by epilithic algae
Infiltration	None, unless designed as open system		H, based on soil Fc	M, based on length	M, if designed as open system	None
TDS Leaching?	Yes	Yes	Yes	No	?	No
Nitrification/ Denitrification?	Nitrification: Yes Denitrification: No		N: Yes D: No	N: Yes D: ?	N: Yes D: ?	N: Yes D: Yes

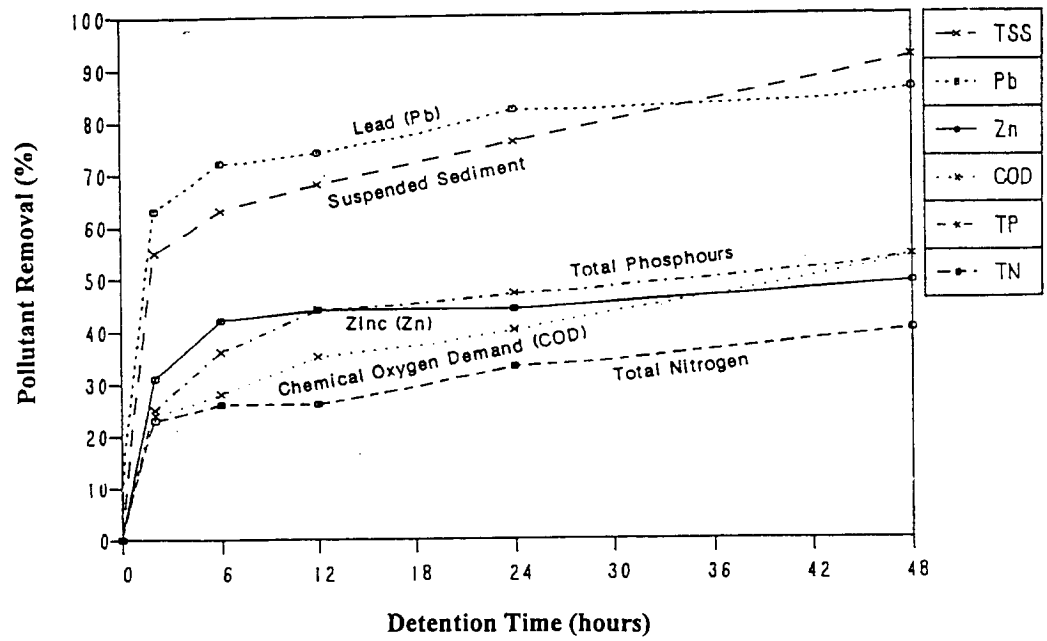
Key: L - Low M - Moderate H - High

TABLE 4.2: THE LIMITS OF SEDIMENTATION: PERFORMANCE OF A DRY EXTENDED DETENTION POND IN NORTH CAROLINA (SOURCE: STANLEY, 1994)

Parameter	*All Storms*	*Big Storms**
Total Suspended Solids	71%	25%
Particulate Organic Carbon	45%	19%
Dissolved Organic Carbon	(-6%)	(-5%)
Cadmium	54%	12%
Copper	26%	11%
Lead	55%	19%
Zinc	26%	11%
Ammonia (NH4-N)	9%	20%
Nitrate-N	(-2%)	6%
Particulate Nitrogen	43%	22%
Total Nitrogen	26%	--
Dissolved Phosphorus	(-9%)	6%
Particulate Phosphorus	33%	17%
Total Phosphorus	14%	--

* Removal rate includes pollutants that bypassed the pond through the emergency spillway and were not subject to settling.

FIGURE 4.1: REMOVAL RATE VS. DETENTION TIME FOR SELECTED POLLUTANTS (SOURCE: GRIZZARD, 1983)



4.1B PATHWAY NO. 2: FILTRATION

Many particulate pollutants are physically strained out as they pass through the filter bed of sand, soil or organic matter, and are trapped on the surface or among the pores of the filter media. The effect of filtration can be very strong. For example, Pitt et al. (1995) report that as much as 90% of small particles commonly found in urban runoff (6 to 41 microns) are trapped by an 18 inch layer of sand, and presumably an even greater percentage of larger particles. As might be expected, the filtration pathway is <u>not</u> effective in removing soluble pollutants and the smallest particles upon which pollutants are often attached. In addition, the importance of the filtration pathway is a function of the media used in the filter. Some of the common chemical properties of several filter media are presented in Table 4.3. In relatively tight media, such as soil or sand, filtration is very important, whereas, in more porous media such as compost or peat, the filtration effect is comparatively weak.

TABLE 4.3
COMPARATIVE PROPERTIES OF DIFFERENT FILTERING MEDIA
(ADAPTED FROM GALLI, 1990; STEWART, 1992 AND PITT et al., 1995)

	Sand	*Silt Loam*	*Compost*	*Peat*
Hydraulic Conductivity (cm/hr)	3.3	0.1 - 0.4		.025 - 140
Water Holding Capacity (cm/cm)	0.14	0.07 - 0.1		.01 - 0.20
Bulk Density (gms/cm)	2.65	1.25 -	1 - 2	<0.1 - 0.3
pH	--	5.7	7.8	3.6 - 6.0
Organic Matter (%)	<1	<20	30 - 70	80 - 98
Cation Exchange Capacity	1 - 3	12 - 18	66	183 - 265
Total-Phosphorus (%)	0.0	0.09	<0.1	<0.1
Total Nitrogen (%)	0.0	0.15	<1.0	<2.5
Filtration Efficiency after 18" (%)	93	94	16	47

4.1c PATHWAY NO. 3: ADSORPTION

The ability of a filtering system to remove soluble nutrients, metals, and organic pollutants is often due to the adsorption pathway, in which ions and other molecules attach to binding sites on filter media particles. In general, the adsorption potential of a filtering system increases when the filtering media has a high content of organic matter or clay, a high cation exchange capacity (CEC) and a neutral to alkaline pH. Once again, each of the media used for filtering systems exhibit sharply different adsorption potentials. Pure sand, for example, initially has little or no organic matter, clay or cation exchange capacity, and therefore, little potential for adsorption (Table 4.3). Over time, however, most sand filters develop a thin layer of organic matter and fine particles at the surface layer of the filter media as a result of sediment deposition thereby increasing the adsorption potential. Organic filter media such as soil, peat and compost, on the other hand, have a much greater potential for adsorption, if the pH of the media is in the optional range.

4.1D PATHWAY NO. 4: INFILTRATION

The bottom of many filtration systems is impermeable so that the filtered runoff can be collected in perforated pipes and returned to the channel. This may be desirable if the contributing site is a hotspot that could contaminate groundwater. If the site is not a hotspot, and underlying soils have a reasonable infiltration rate, however, it is possible to utilize infiltration as a major removal pathway. Both runoff and entrained pollutants can migrate downward into the soil layer (which can provide additional filtering or adsorption of pollutants), and may eventually reach the water table. Runoff infiltration is a major removal pathway for several filtering systems, such as dry swales, filter strips and some bioretention designs. In many cases, the total mass removal of these systems is proportional to the mass of pollutants that are infiltrated into the soil.

4.1E PATHWAY NO. 5: MICROBIAL ACTION

Filter media are inevitably colonized by microbes that break down organic pollutants, and transform nutrients. Optimal growth of microbes is achieved when organic matter is plentiful, temperatures are warm and the filter media moist. "Biofilms" develop around filter particles, providing an ideal surface area for microbial growth. Microbial action is a significant pollutant removal pathway in most filtering systems, and two microbial processes in particular, are very important in explaining their nitrogen dynamics—nitrification and denitrification.

NITRIFICATION

Nitrification is an important nitrogen removal pathway as organic matter is gradually decomposed. Microbes break down organic nitrogen into ammonia, which is then transformed into soluble nitrate-nitrogen. The nitrification process generally requires an aerobic (oxygen-rich) environment which is characteristic of many filtering systems. As a result, nitrification occurs rapidly in many filtering systems, resulting in the export of low concentrations of ammonia.

DENITRIFICATION

The final step in the nitrogen cycle is the conversion of soluble nitrate into nitrogen gas that is returned to the atmosphere. To proceed, the denitrification process requires a moist, anaerobic environment (zero-oxygen), an abundant supply of both organic carbon and nitrate, and the presence of denitrifying bacteria. These conditions are not always met in most filtering systems. Consequently, most filtering systems actually export more soluble nitrate than they receive. In recent years, designers have attempted to create suitable conditions for denitrification within filtering systems, and have demonstrated a capability to remove nitrate.

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4.1F PATHWAY NO. 6: PLANT RESISTANCE AND UPTAKE

Several filtering systems incorporate plants, such as algae, emergent wetlands or grass to improve removal rates. Examples include vegetated open channels (grass), sand or organic filters (that have a grass cover crop), bioretention, filter strips, and gravel wetland filters (algae, wetland plants). Plants can increase pollutant removal in several ways. During periods of stormflow, for example grass and emergent wetland plants provide resistance to flow, thereby reducing runoff velocities. Slower runoff velocities translate into more time for other pollutant pathways to work (such as settling, filtering, infiltration and adsorption). In addition, the roots of grass and emergent plants help bind up the filter media, preventing loss of sediments and attached pollutants via erosion.

The growing plants also create a continual supply of thatch or detritus, which provide the organic matter needed for greater adsorption. During periods of growth, the plants also take up nutrients and metals from the filter bed and incorporate it into their biomass. If plant biomass is harvested or mowed, pollutants are removed. Taken together, however, the use of plants in a filtering system is usually of secondary importance as a pollutant removal pathway in comparison to the other five pathways.

4.2 PERFORMANCE MONITORING STUDIES

In this section, nearly forty performance monitoring studies of stormwater filtering systems are reviewed, in order to extract general principles with regard to pollutant removal that can be used in design. As with any broad review of monitoring studies, it is important to keep in mind some important caveats with respect to their interpretation. The monitoring studies were not carefully controlled replicates. Instead, they encompassed a wide geographic and climatic range, reflect considerable differences in basic designs, utilized different methods to compute pollutant removal, and exhibited wide differences in inflow and outflow capture, storm bypass, and number of storms sampled. Even with these many differences among the studies, however, several important generalizations can be made with respect to pollutant removal performance of filtering systems.

4.2A POLLUTANT REMOVAL PERFORMANCE OF SAND FILTERS

Presently, performance monitoring data for sand filters consists of nine studies conducted in Austin TX, Seattle WA, Orlando FL, and Alexandria VA. In addition, one compost filter has been extensively monitored (Table 4.4).

Table 4.4
Pollutant Removal Performance of Sand Filters

No.	Reference	State	YR	N	M	Type	A	TSS	OC	TP	SP	TN	NO3	Cu	Pb	Zn	Other
1	WELBORN	TX	87	22	M	surface sand filter	80.0	78	66	27	-	27	(-111	-	33	60	B=81
2	AUSTIN ERM	TX	91	18	M	surface sand filter	80.0	75	45	59	-	44	(-13)	34	88	82	B=36
3	AUSTIN ERM	TX	91	17	M	retention-surface sand	50.0	92	85	80	-	71	23	84	89	91	B=83
4	AUSTIN ERM	TX	91	16	M	off-line surface sand	10.0	87	57	61	-	32	(-79)	60	81	80	B=37
5	AUSTIN ERM	TX	91	18	M	on-line surface/grass	3.0	86	41	19	-	31	(-5)	33	71	49	B=37
6	BELL	VA	95	20	M	perimeter sand filter	1.0	79	66	65	-	47	(-53)	25	N/A	91	-
7	HORNER	WA	95	20	M	perimeter sand filter	2.0	83	-	46	-	-	-	-	22	33	TPH=84
8	HORNER	WA	95	20	M	perimeter sand filter	2.0	8*	-	20	-	-	-	31	-	69	TPH=55
9	HARPER	FL	93	33	M	retention/sand filter	ND	98	99	61	(-37)	0	27	37	71	89	TKN=0
10	EGAN	FL	95	12	M	packed bed filter	120.0	81	38	82	14	63	75	21	73	55	-
11	REUTER	CA	93	15	C	gravel bed filter	3.0	84	-	30	(-35)	NEG	NEG	-	-	-	-
12	STEWART	WA	94	15	M	compost filter	74.0	92	-	45	(-197	-	(-144	65	83	83	TPH=85
13	GALLI	-	92	est.	-	peat sand filter	-	[90]	[90]	[70]	-	[50]	-	[80]	[80]	[80]	B=90
								85	62	50	NEG	35	NEG	43	68	71	TPH=75
								Mean removal rate; excluding #13									B=55

Notes: N=no. of storms sampled, M=method used to compute removal (M=mass, c=concentration), A=drainage area (acres), OC=Organic carbon (COD/BOD or TOC), B=fecal coliforms, NEG=negitive. *low removal due to very low input concentrations

The initial monitoring results suggest that sand filters are very effective in removing particulate pollutants, such as total suspended solids, lead, zinc, organic carbon and organic nitrogen, but exhibit rather mediocre removal of soluble pollutants. Removal rates for coliform bacteria, ammonia, ortho phosphorus and copper were moderate, and quite variable—ranging from 20 to 75% in the ten sand filters tested.

SUSPENDED SEDIMENT

Sand filters uniformly demonstrated an excellent ability to remove suspended sediment. Mean TSS removal rates of 75 to 90% were reported at most sites. The high sediment removal is expected, given the effectiveness of filtration and settling in removing particulates. It is not entirely clear what proportion of the removal occurs in the filter bed (filtration) or in the pretreatment chamber (settling), but the two pathways appear complimentary. Pitt et al. (1995), however, reported in settling column studies indicated that some very fine sediment particles (10 microns or less) may not be captured in the sand filter, and pass through.

ORGANIC CARBON

Sand filters were found to be effective in removing various forms of organic carbon (BOD, COD and TOC) with mass removal in most sand filters ranging from 45 to 65%. Settling and filtration again were the dominant pollutant removal pathways for organic carbon. It should be noted, however, that some particulate organic carbon deposited or trapped on the filter bed decomposes, and may be actually exported from the filter as dissolved organic carbon.

NUTRIENTS

Most sand filters showed a moderate capability to remove total nitrogen, with an average removal rate of about 35%. Of the nitrogen forms that comprise total nitrogen, the greatest removal is noted for TKN (organic nitrogen). A sizeable fraction of TKN is in particulate form, making it susceptible to settling and filtration. Removal of soluble nitrate-nitrogen is usually negative, indicating that while nitrification is occurring in the filter media, denitrification is not.

The nine sand filters consistently exhibited a moderate to high potential to remove total phosphorus, with six filters exceeding 60%. Not surprisingly, much of the removal rate can be ascribed to the settling and filtration of the particulate fraction of phosphorus (which is often about 50 or 60% of total phosphorus). Removal data on the more biologically available forms of phosphorus (ortho or soluble reactive) are more limited. Bell et al. (1995) reports high levels of removal (Table 4.5).

TABLE 4.5: POLLUTANT REMOVAL OF THREE DELAWARE SAND FILTERS
(SOURCE: BELL et al. 1995, HORNER AND HORNER 1995)

Parameter	Alexandria, VA Bell et al., 1995 Mass Removed (a)	Seattle, WA Horner and Horner, 1995 Mean Removal (b)	Seattle, WA Horner and Horner, 1995 Mean Removal (b)
Number of Storms	20	14	6
Total Suspended	79%	83%	8% (c)
Oil and Grease	NA	84%	69%
Petroleum	ND	84%	55%
Total Organic Carbon	66%	NA	NA
BOD (five-day)	78%	NA	NA
Total Phosphorus	63% (d)	41%	20%
Ortho-Phosphorus	68% (d)	NA	NA
Total Nitrogen	47%	NA	NA
Nitrate+Nitrite	(-53.3%)	NA	NA
TKN	70.6%	NA	NA
Zinc	91%	33%	69%
Copper	25% (b)	22%	31%

Notes:
- **(a) fraction of total incoming pollutant load retained in filter over all storms**
- **(b) average of storm pollutant concentration reduction, all storms**
- **(c) poor removal due to very low TSS inflow concentrations (4 - 24 mg/l)**
- **(d) removal rates were higher if four anaerobic events are excluded**
- **NA parameter not analyzed during monitoring study**
- **ND parameter not detected in runoff during sampling study**

TRACE METALS

Sand filters were usually capable of removing trace metals, such as lead and zinc with an average rate of about 70%. High removal is generally expected for these metals, since they are often attached to particles that easily settle or filter out. Sand filters showed less ability to remove metals predominantly found in soluble form. For example, Bell et al. (1995), Horner and Horner (1995) and Austin ERM generally reported copper removal on the order of 20 to 35%. The fact that any soluble metals were removed was surprising given that pure sand has virtually no adsorptive capacity.

BACTERIA

Five sand filters were analyzed to determine their ability to remove fecal coliform bacteria. Bacteria removal ranged from 37 to 83%. The relatively modest bacteria removal noted in the five Texas filters was surprising, given that sand filters have been used extensively to treat drinking water, and typically remove 90 to 95% of incoming bacteria (Ellis, 1987).

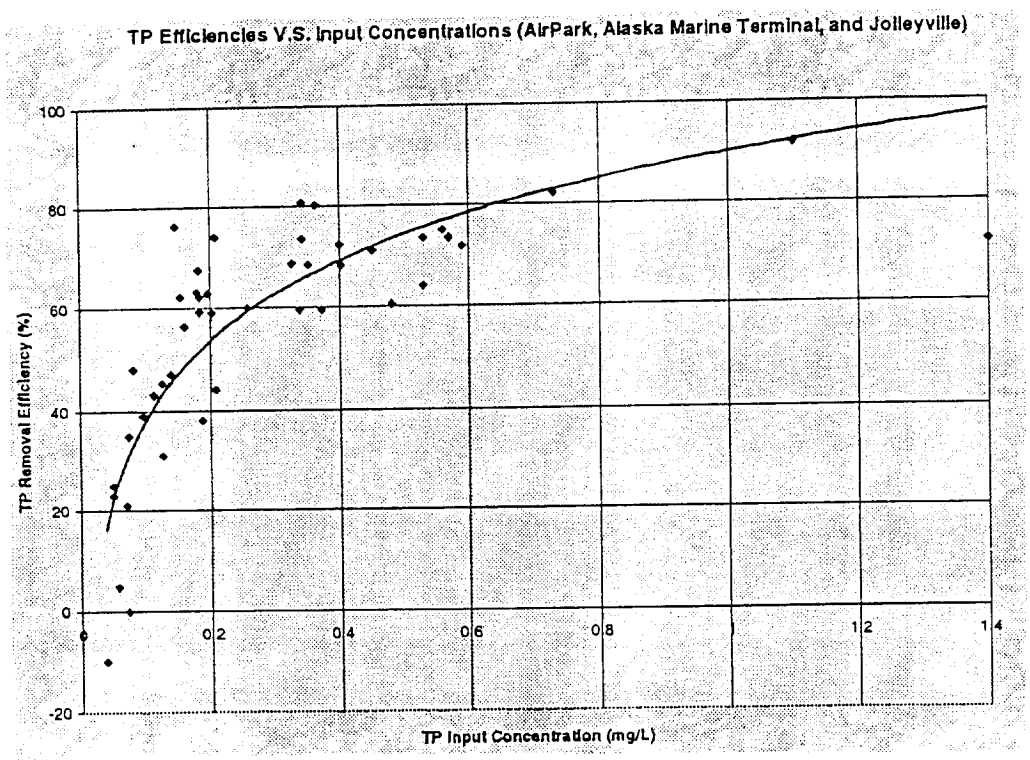
PETROLEUM HYDROCARBONS

Horner and Horner (1995) examined the ability of two sand filters to remove petroleum hydrocarbons and oil and grease at a Seattle marine terminal, and reported removal rates that ranged from 55 to 84%. He also detected a strong dose response relationship. When incoming hydrocarbon concentrations were low, for example, removal rates were relatively low. However, when incoming hydrocarbon concentrations surpassed 3 mg/l, removal rates consistently exceeded 90%. This suggests that sand filters may be an effective practice for controlling hydrocarbons at stormwater pollution hotspots.

DOSE-RESPONSE RELATIONSHIP

Bell conducted a detailed analysis of the relationship between inflow concentration and pollutant removal on sand filters that were monitored in Alexandria, Seattle and Texas. He detected a strong relationship between inflow concentration and removal efficiency for sediment, phosphorus, organic nitrogen, zinc, and total petroleum hydrocarbons. Simply put, removal efficiency sharply increased when the concentration of particulate pollutants entering the sand filter was high, and dropped when incoming pollutant concentrations were low (and of less water quality significance). The dose-response relationship for total phosphorus is depicted in Figure 4.2.

FIGURE 4.2: DOSE-RESPONSE RELATIONSHIP FOR TOTAL PHOSPHORUS IN SAND FILTERS (SOURCE: BELL et al. 1995)



IRREDUCIBLE CONCENTRATIONS FROM SAND FILTERS

After analyzing the effluent quality of many monitored BMPs, Schueler (1996) has shown that there is an apparent minimum pollutant concentration that is always discharged from sand filters, as well as other BMPs. Pollutant levels cannot be reduced below this rather low level using existing technology. The irreducible concentration computed for six to ten sand filter systems that reported outflow concentrations are provided in Table 4.6. The approximate limits to sand filter treatment for some common urban pollutants are shown below.

- TSS 20 mg/l
- TN 2.0 mg/l
- TP 0.15 mg/l

The irreducible concentration may represent leaching or flushing of pollutants that had been trapped on the filter surface over time. It should be noted that the irreducible concentration values for sand filters are surprisingly comparable to those derived at pond and wetland systems (Schueler, 1996).

TABLE 4.6: IRREDUCIBLE CONCENTRATIONS OF SAND AND ORGANIC FILTERS (SOURCE: SCHUELER, 1996)

Parameter	*N*	*Concentration (mg/l)*
Total Suspended Solids	10	19.3 ± 10.1
Total Phosphorus	10	0.14 ± 0.13
Ortho-Phosphorus	ND	--
Total Nitrogen	6	1.93 ± 1.02
Total Kjeldahl Nitrogen	6	0.90 ± 0.52
Nitrate-Nitrogen	6	1.13 ± 0.55
Data Sources: Horner and Horner (1995), City of Austin (1990), Bell et al. (1995), CSF (1994)		

NITRIFICATION EFFECT

Nitrate export was observed in five out of seven sand filters monitored for parameter. This behavior suggests that nitrification is taking place within the filter bed. During the nitrification process, microbial bacteria converts ammonia-nitrogen into the nitrate form of nitrogen. The apparent loss of ammonia through the filter bed, coupled with the production of excess nitrate, strongly suggests that nitrification is taking place. Nitrate export has also been observed in other stormwater filtering systems that do not rely on sand (i.e., compost and grass channels). Apparently, sand filter conditions do not allow for significant denitrification to occur (that converts nitrate into nitrogen gas). Bell has speculated that denitrification could be more pronounced if the bottom of the sand filter is allowed to become anaerobic (i.e., designing lengthy periods of water saturation in the bottom layer) and presents some evidence that denitrification did occur at several microsites in his Alexandria test filter.

LEACHING EFFECT

Negative removal rates were frequently reported for total dissolved solids (TDS) and nitrate-nitrogen, and occasionally, for soluble phosphorus and metals. The negative TDS rate may be due to the preferential leaching of cations from organic matter trapped on the surface of sand filter. The leaching effect was observed regardless of whether the filter medium was sand or compost. Huang and Petrovic (1994) noted that a layer of zeolites could retain nitrate and other cations from leaching through the sand layer below a golf course green.

ALTERNATIVE MEDIA

Limited monitoring data are presently available to assess whether organic media are more effective than sand in trapping pollutants. Pitt et al. (1995) noted that experimental sand columns did not always securely retain small sediment particles that contained toxicants and metals, but often flushed them through the filtering column, and has investigated whether peat, compost, activated carbon or soil would increase retention of smaller particles. He notes that organic media in combination with sand have considerable potential to increase removal of a sand filter.

At this time, the pollutant removal performance of only one alternative media filter system has been monitored in the field (CSF, 1994). The compost filter system provided excellent removal of sediment, particulate nutrients, organic carbon, hydrocarbons and some trace metals (Table 4.7). Total dissolved solids, however, increases, which appears to reflect the exchange and/or leaching of cations within the compost. Similarly, while particulate nutrient forms are trapped within the compost, the system exports soluble forms of nutrients, such as nitrate and soluble phosphorus. The organic matter in the compost has a high cation exchange capacity and therefore has a greater potential to adsorb soluble metals and organics.

TABLE 4.7: POLLUTANT REMOVAL PERFORMANCE OF A COMPOST FILTER (SOURCE: CSF, 1994)

Parameter	*% Removed*
Total Suspended Solids	95%
Total Dissolved Solids	(-37%)
COD	67%
Total Phosphorus	41%
Soluble Phosphorus	(negative)
Organic Nitrogen	56%
Nitrate	(-34%)
Cadmium	ND
Lead	ND
Zinc	88%
Hydrocarbons	87%
Copper	67%
Boron, Calcium,	(negative)

COMPARISON WITH WASTEWATER SAND FILTER PERFORMANCE

The pollutant removal behavior of stormwater sand filters is generally comparable to that reported for sand filters used in wastewater treatment (Ellis, 1987). Wastewater sand filters typically contain finer sand, are cleaned more frequently, and are subject to more uniform and controlled flow than their stormwater counterparts. Consequently, wastewater filters exhibit slightly higher removal rates for sediment, phosphorus and organic carbon (often in excess of 90%), but seldom can achieve more than 20% removal of nitrate (again, due to the lack of denitrification). They do show greater capability to remove fecal coliform bacteria.

4.2B OPEN VEGETATED CHANNELS

Few best management practices exhibit such a great variability in pollutant removal performance as open grass channels. Sixteen historical performance monitoring studies of "grass swales" were re-analyzed based on the open channel classification presented earlier to try to explain this variability. Ten of the open channels could be classified as "drainage channels" based on two criteria—they were designed only to be non-erosive for the two year storm, and their particular combination of soil and slope did not allow significant infiltration of runoff into the soil profile. Site data and pollutant removal data for these drainage channels are shown in Table 4.8. The poor performance of drainage channels is due to the fact that they do not act as an effective filter (i.e., very little runoff actually filters through the soil media). Since the soil filter is not used, drainage channels can only rely on sedimentation and adsorption pathways for removal. During most storms, runoff passes through the channel in just a few minutes, thereby greatly reducing the effectiveness of those removal pathways.

One open channel was explicitly designed as a grassed channel (Seattle METRO , 1992). The 200 foot long grass channel, termed a biofilter, was found to be reasonably effective in removing many pollutants contained in urban stormwater. The performance monitoring data for the biofilter is summarized in Table 4.9. In general, high rates of removal were reported for sediment, hydrocarbons, and particulate trace metals. Nutrient removal was much more mixed.

Five open channels were either explicitly designed as a dry or wet swale, or had a combination of soils, slope and water table so that they effectively functioned like one (Table 4.10). Given the small number of open channels that met these criteria, they were lumped together as a single group. The swales demonstrated a much greater and more consistent capability to remove pollutants conveyed in urban stormwater. In nearly every case, most of the mass removal could be accounted for by the infiltration or retention of runoff into the soil profile during storms (i.e., actual pollutant concentration did not change appreciably as they passed through the channel). As a group, the swales showed excellent removal of suspended sediment, nitrogen, organic carbon and trace metals.

Table 4.8
Pollutant Removal Performance of Ten Drainage Channels

No	Reference	State	YR	N	M	S	L	A	SOIL	TSS	OC	TP	SP	TN	NO3	Cu	Pb	Zn	Other
1	OWML	VA	83	33	M	1.8	260	9.5	SL	NEG	NEG	NEG	-	NEG	-	-	NEG	NEG	-
2	OWML	MD	83	50	M	4.1	445	19.0	SL	NEG	NEG	NEG	-	NEG	-	-	NEG	NEG	-
3	OWML	MD	83	8	M	5.1	425	12.0	SL	31	NEG	NEG	-	37	-	-	33	NEG	-
4	DORMAN	VA	89	9	M	4.7	185	1.3	SL	65	76	41	-	-	11	28	48	49	TKN=17
5	DORMAN	MD	89	4	M	3.2	193	1.3	SL	NEG	23	12	-	-	NEG	14	55	9	TKN=9
6	YU	VA	89	4	M	5A	200	1.5	-	68	-	60	-	-	-	-	-	74	-
7	YOUSEF	FL	85	6	C	1.0	550	-	Sa	-	-	8	26	13	11	14	27	29	TKN (-20)
8	OAKLAND	NH	83	11	C	>2%	100	-	-	33	-	NEG	NEG	-	-	48	57	50	COLI = NSD
9	WELBORN	TX	87	19	C	-	200	2.9	-	NSD	NEG	NEG	NEG	NEG	NEG	NSD	NSD	NSD	COLI = NSD
10	PITT	ONT	86	50	C	-		-	-	NSD	-	-	-	NSD	-	NSD	NSD	NSD	COLI = NSD

Notes. N=number of samples, M=mass or concentration method, S=slope, L=length, A=contributing area (acres), SOIL (SL=silt loan, Sa=sandy), COLI=fecal coliforms, NEG=negative removal efficiency reported, NSD=no statistically different concentration between control (usually pipe flow)

TABLE 4.9: POLLUTANT REMOVAL PERFORMANCE OF A GRASS CHANNEL (BIOFILTER) OF TWO LENGTHS IN WASHINGTON (SOURCE: SEATTLE METRO, 1992)

Pollutant	*100 Foot Biofilter*	*200 Foot Biofilter*
Suspended Sediment	60%	83%
TPH (Hydrocarbons)	49%	75%
Total Zinc	16%	63%
Dissolved Zinc	negative	30%
Total Lead	15%	67%
Total Copper	2%	46%
Total Phosphorus	45%	29%
Bioavailable P	72%	40%
Nitrate-N	negative	negative
Bacteria	negative	negative

TSS

Only four out of nine drainage channels had a positive removal rate for suspended sediment, suggesting that neither settling, filtration or infiltration occurred to any great degree as it passed through the channels. By contrast, sediment removal rates for dry swales, wet swales and the grass channel all exceeded 80%.

ORGANIC CARBON

Drainage channels showed little ability to remove organic carbon, with four of six tested showing negative removal rates. Both dry swales and wet swales on the other hand, had carbon removal rates in excess of 50%. While no data was available for grass channels it would appear reasonable that settling and filtration pathways would be effective for this primarily particulate pollutant.

Table 4.10
Pollutant Removal Performance of Six Water Quality Swales

No.	Reference	State	YR	N	M	S	L	A	SOIL	TSS	OC	TP	SP	TN	NO3	Cu	Pb	Zn	Other
1	DORMAN	FL	89	8	M	1.0	185	0.6	Sa	98	64	18	-	-	45	65	81	81	TKN=48
2	HARPER	FL	88	16	M	1.0	210	0.8	Sa	87	69	83	-	84	80	89	90	90	-
3	HARPER	FL	88	11	M	1.8	210	1.2	WET	81	48	17	-	40	52	56	50	69	-
4	KERCHER	FL	83	13	M	>2.0	-	14.0	Sa	99	99	99	-	99	99	-	99	99	
5	METRO	WA	92	6	C	4.0	200	16.0	till	83	-	29	72	-	NEG	46	67	73	HC=75 COLI=NEG
6	WANG	WA	81	8	M	-	200	-	-	80	-	-	-	-	-	70	80	60	-

Notes. N=number of samples, M=mass or concentration method, S=slope, L=length, A=contributing area (acres), SOIL (SL=silt loan, Sa=sandy), HC=hydrocarbons, COLI=fecal coliforms, NEG=negative removal efficiency reported, NSD=no statistically different concentration between control (usually pipe flow)

NUTRIENTS

Drainage channels provided negligible removal of nutrients. In most sites, nitrogen and phosphorus removal was either consistently low or non-existent. Nutrient removal in the grassed channel, in contrast, was somewhat higher, with about 30% of total phosphorus and 70% of soluble phosphorus effectively removed (Seattle METRO, 1992). The grass channel was also a net exporter of nitrate.

Dry and wet swales showed better ability to remove nitrogen, with the mass removal rates ranging from 40 to 99%. Phosphorus removal was more variable, with the two swales experiencing the most infiltration recording phosphorus removal rates greater than 80%, and three reporting with minor infiltration capability showing removal rates of 30% or less. Phosphorus removal may be limited in any open channel system. Monitoring has shown that open channels have high phosphorus levels stored in the thatch and surface soil layer. Some of the stored phosphorus may recycle back into the water column, or be eroded during larger storms. In addition, the high phosphorus levels in channel soils may be too high to allow meaningful adsorption.

TRACE METALS

While some drainage channels did exhibit a moderate ability to remove trace metals attached to particles (i.e, lead and zinc), an equal number showed no metal removal capability whatsoever. By contrast, trace metal removal rates for grass channels, dry swales and wet swales were uniformly high. It should be noted that most metal removal is due to settling and filtering of metals attached to particles. Removal of soluble metals, however, was only 20 to 50% (Yousef et al., 1985).

Most monitoring studies only report removal of total trace metals, and do not independently measure the fraction of metals found in soluble form. This can be significant as soluble metals usually exert the greatest impact or toxicity to aquatic life. Many trace metals are primarily found in soluble forms (cadmium, copper and zinc), while others are mostly attached to sediment particles (iron and lead). Yousef et al. (1985) found that swales were not very effective at adsorbing soluble metal species. Adsorption requires that a metal be present in runoff as a positively charged cation that can be adsorbed to a negatively charged particle in the soil or organic layer. Metals, however, can be found in a complex number of ion species depending on the prevailing acidity (pH) of runoff. Some metals such as zinc readily adsorb to soil at pH levels typical of stormwater runoff 6.5 to 8.0, but many others (aluminum, cadmium, copper, chromium and lead) show little tendency to adsorb to soils within this pH range. Consequently, the ability of swale soils to remove many soluble trace metals tends to be rather low.

BACTERIA

The three studies that examined the ability of drainage channels to remove fecal coliform bacteria found no significant change in the counts of this key human health indicator after channel treatment. Oakland (1983), Welborn and Veenhuis (1987), Pitt and McLean (1986) all reported that drainage channels had no effect in reducing bacterial concentrations as they traversed through the swale. Seattle METRO (1992) also reported that a grass channel actually tended to increase the level of fecal coliform bacteria as runoff passed through it. This increase was thought to be due pet droppings and possible bacterial multiplication within the biofilter itself.

PETROLEUM HYDROCARBONS

The only study that examined hydrocarbon removal in grass channels found they were very effective at removing both hydrocarbons and oil and grease (Seattle METRO, 1992).

CHLORIDES

Open channels appear to have no capability to trap soluble chlorides (Harper, 1988, Demers and Sage, 1990).

METAL AND NUTRIENT ACCUMULATION IN SOILS

A number of researchers have found that both metals and nutrients tend to be higher in surface soils of open channels than adjacent upland soils. (Wiggington et al. 1983, Dorman et al. 1989, Harper 1988, WCC 1994, Lind and Karro 1995). A summary of the average concentration of metals and nutrients in twelve open channel systems in the U.S. can be found in Table 4.11. The higher levels appear to suggest that swales are accumulating metals and nutrients. One interpretation from the data might be that open channels are trapping and retaining these pollutants, but it can also be argued that swales are simply a better depositional environment. Since swales are a depression in the landscape, they represent an excellent depositional site for aerosols and dust generated by vehicles on adjacent roads, and this factor may well explain the higher levels.

Another interesting aspect of Table 4.11 is the surprising consistency in phosphorus, organic nitrogen, copper and zinc levels in surface soils among the many geographically diverse sites. The only pollutant that exhibits great variability is lead. The lead variability may be due to the declining rates of lead deposition in recent years associated with the gradual introduction of unleaded gasoline, and localized differences in airborne lead deposition due to traffic factors.

According to Lind and Karro (1995), soil type is very important factor for metal accumulation in open channels. Those that have a high content of clay or organic matter in surface soils are able to adsorb metals better.

TABLE 4.11: SEDIMENT POLLUTANT LEVELS IN TWELVE GRASS DRAINAGE CHANNELS (ALL VALUES IN MG/KG)

Reference	*State*	*TP*	*TKN*	*NO_3*	*Copper*	*Lead*	*Zinc*
Dorman	VA	1057	947	0.5	39	100	106
Dorman	MD	1135	1794	10	32	419	251
Dorman	FL	1112	1900	12	11	143	144
Wiggington	VA	-	-	-	4	42	101
Wiggington	MD	-	-	-	10	17	70
Wiggington	VA	-	-	-	23	936	106
Harper	FL	748	1524	-	75	1378	680
Harper	FL	571	1971	-	22	325	157
WCC	CA	-	-	-	36	262	225
WCC	CA	-	-	-	37	43	142
WCC	CA	-	-	-	43	82	179
WCC	CA	-	-	-	20	11	85
Mean		**924**	**1627**	**8**	**29**	**313**	**187**

CULVERT LEACHING

Wiggington et al. (1986) discovered that galvanized metal culverts that are often used for driveway crossings in residential swale systems can be a source of some trace metals. Under some conditions, the metal coating of these pipes leach trace metals, particularly when runoff is slightly acidic. The leaching effect was most pronounced for zinc, but was also observed for copper and cadmium.

INFILTRATION

One of the key benefits of dry swales is their ability to reduce the volume of runoff through soil infiltration. Pitt and McLean (1986) noted that while pollutant concentrations did not change through open channels in metropolitan Toronto, they did produce 25% less annual runoff. This effect was particularly evident for storms smaller than a half inch. Anderson (1982) and Yu et al. (1992) also observed that swales seldom produced measurable runoff during storms, although adjacent curb and gutter systems did. The importance of the infiltration pathways in dry swales is evident in the work of Yousef et al. (1985). As can be seen in Table 4.12, the total mass removal through the test channel was roughly proportional to the mass of runoff that fully infiltrates through the bottom of the channel. Again, pollutant concentrations in runoff that did not infiltrate through the channel bottom did not change appreciably in Yousef's study.

TABLE 4.12: NUTRIENT REMOVAL IN SIX EXPERIMENTAL SWALES IN FLORIDA AS A FUNCTION OF RUNOFF VOLUME INFILTRATED (SOURCE: YOUSEF ET AL., 1985)

Site Number	*Infiltration Volume*	*Nitrate-N*	*Organic-N*	*Total N*	*Diss. P*	*Total P*
M-6	26%	(-2%)	22%	27%	26%	31%
E-4	38%	48%	41%	39%	43%	45%
E-5	50%	(-21%)	41%	24%	40%	27%
M-1	57%	57%	64%	61%	62%	63%
M-2	60%	67%	63%	73%	79%	79%
M-3	100%	100%	100%	100%	100%	100%

SOLUBLE NUTRIENTS

The channels experiments conducted by Yousef et al. (1985) indicated that most swales showed little capability to adsorb or filter soluble forms of nitrogen and phosphorus as they passed through the swale. Little or no reduction in soluble nutrient concentration was observed. The bulk of the mass nutrient removal in the

channel could be accounted for by simple infiltration of runoff through the bottom of the swale. Indeed, a cursory glance at Table 4.12 shows that total removal rates and the fraction of total runoff infiltrated into the swale bottom were essentially identical. This implies that the major pollutant removal pathway in dry swales is an underground one (infiltration) and not necessarily a surface one (settling, filtering or adsorption).

IRREDUCIBLE CONCENTRATIONS

Only a small number of drainage channels and dry swales reported outflow data from which the irreducible concentration could be computed (Table 4.13). The provisional values for the limits of open channel treatment for some common pollutants are provided below. Please note that these values have a considerable standard deviation.

- TSS 40 mg/l
- TP 0.30 mg/l
- TN 1.75 mg/l

With the exception of total nitrogen, open channels appear to have a higher "irreducible concentration" for sediment, total phosphorus and soluble phosphorus than other BMP systems (ponds, wetlands, and sand filters).

TABLE 4.13: ESTIMATED IRREDUCIBLE CONCENTRATION OF OPEN CHANNEL SYSTEMS (SOURCE: SCHUELER, 1996)

Parameter	*N*	*Concentration*
Total Suspended Solids	5	43.4 ± 47.0
Total Phosphorus	5	0.33 ± 0.15
Ortho-Phosphorus	3	0.16
Total Nitrogen	5	1.74 ± 0.71
Total Kjeldahl Nitrogen	5	1.19 ± 0.41
Nitrate-Nitrogen	5	0.55 ± 0.29
Data Sources: Harper (1988) and Dorman et al. (1989)		

LENGTH/CONTACT TIME EFFECT

Dorman et al. (1989) concluded that channel length alone was not a reliable predictor of the removal efficiency in drainage channels. Although the ten drainage channels ranged in length from 100 to 550 feet, there was no relationship between length and removal efficiency. A quick calculation illustrates why channel length, by itself, is not useful parameter. Given a typical stormflow velocity of 1.5 feet per second, it takes just over a minute to travel 100 feet of a channel (which allows very little time to utilize adsorption, settling or infiltration pathways).

The grass channel design alters the geometry of the channel to decrease the speed of runoff. Seattle METRO (1992) reported that a 10 minute residence time in a grass channel is needed to attain reliable pollutant removal for most storms. Their monitoring indicated that a 200 foot grass channel did perform better than a 100 foot grass channel.

SOIL TYPE

Soil type is an important design factor for three reasons. First, the soil type governs the rate of infiltration that can occur. A sandy soil, for example, often allows for substantial infiltration of runoff, whereas a clay soil does not. Consequently, many dry swales utilize natural or prepared "sandy" soils to infiltrate significant runoff volumes. Second, soil type is influential in determining the rate of adsorption. Soils with a high clay or silt content and soils with a high organic matter content have a higher adsorption potential than sandier ones (See Table 4.2). Third, the underlying soil type often determines the density and vigor of grass cover in the swale. Extremely clayey or sandy soils often make it difficult to establish the vigorous grass cover needed to provide flow resistance and prevent channel erosion.

4.2C VEGETATED FILTER STRIP

Our current knowledge about the pollutant removal capability of urban vegetated filter strips is confined to a single study. Yu et al. (1993) analyzed a grass filter strips to treat urban stormwater runoff from a large parking lot (Table 4.14). Yu reported moderate to high removal rates for a 150 foot grass strip, and mediocre pollutant removal performance in a shorter, 75 foot strip.

Most of the research on the pollutant removal capability of filter strips has been conducted in agricultural areas. Desbonette et al. (1994) has conducted an excellent review of nearly 35 different agricultural monitoring studies. The buffer studies can be grouped into two categories: those that utilize grass filter strips to treat sediment and nutrient laden surface runoff from row crops, and those that employ forested strips to remove nutrients in subsurface flows from crop and pastureland.

Moderate removal rates were consistently reported for sediment, nitrogen and

phosphorus for filters that treat surface runoff (Desbonette et al., 1994). Typically, a fifteen foot wide grass buffer can achieve a 50% reduction in all three pollutants in surface runoff. Further increases in the removal rate, however, require substantially higher filter widths. For example, an average of 70% removal for all three pollutants was attained when the strip length is increased to 100 feet. Impractically long strips (300 to 600 feet) are needed to attain a consistent 90% removal rate for these pollutants.

TABLE 4.14: POLLUTANT REMOVAL OF AN URBAN VEGETATED FILTER STRIP IN VIRGINIA (SOURCE: YU ET AL., 1993)

Parameter	Removal Performance of LS/VBS System	
	75 Foot Filter Strip	150 Foot Filter Strip
Total Suspended Solids	54%	84%
Nitrate+Nitrite	(-27%)	20%
Total Phosphorus	(-25%)	40%
Extractable Lead	(-16%)	50%
Extractable Zinc	47%	55%

The ability of grass and forested buffers to remove nutrients in subsurface flow has been mixed. When conditions are ideal, very high removal rates for nitrate have been reported. These conditions include the combination of poorly drained and highly organic soils, trees with deep roots systems, and lateral groundwater movements within four to six feet of the surface. These conditions actively promote the denitrification process, which converts nitrate-nitrogen into nitrogen gas. Where such conditions persist, forest buffers can effectively reduce the nitrogen content of septic system effluent in rural residential areas.

In general, most researchers consider agricultural filter strips to be a useful BMP, but only when they are combined with other practices (Magette et al. 1989). It is also widely recognized that many agricultural filter strips fail to perform as designed after they are installed in the field (Dillaha et al., 1989). Field surveys indicate that many filter strips lack good vegetative cover, are subject to excessive sediment deposition, or are short-circuited by channels formed by concentrated flow. This is particularly true for filter strips employed in urban areas, where runoff concentrates very quickly.

4.2D BIORETENTION

No monitoring data is presently available to assess the pollutant removal capability of bioretention areas. Based on the number and redundancy of possible removal pathways, it is very likely that the removal rate will be high. In many ways bioretention areas function in the same manner as a dry swale (i.e., both filter ponded runoff through a filter bed of prepared soil), so it is presumed that their pollutant removal capability would also be similar to the dry swale.

4.2E SUBMERGED GRAVEL FILTERS

Three submerged gravel or rock filters have been monitored to determine their ability to remove pollutants in stormwater runoff (Egan et al.,1995, Horsley,1995 and Reuter et al., 1992). Although both the design and site conditions associated with each filter were very different, the performance of the submerged gravel filters as a group appears to be very promising.

Egan designed and constructed an experimental "stormwater treatment train" to treat runoff from a 121 acre industrial subwatershed in Central Florida. The off-line system featured packed bed filter cells. Each packed bed filter cell was excavated into the soil, and had dimensions of 80 feet wide by 30 feet long and three feet deep. The bottom of each cell was sealed with a plastic liner, and then filled with either crushed concrete or granite rock. Eight filter cells were planted with one or more of the following emergent wetland plant species: maidencane, giant bulrush, fireflag. Two cells were not planted to serve as controls (i.e., to test the pollutant removal capability of the rock media itself). The overall pollutant removal performance of Egan's packed bed filter system is summarized in Table 4.15.

Horsley's design is termed the StormTreat system and consists of a circular tank. Runoff passes through internal sedimentation chambers for pretreatment, and then is diverted into a outer ring that is filled with a gravel/sand media in which wetland plants are rooted. A schematic of the StormTreat system is provided in Figure 4.3, and recent performance monitoring data for an experimental site in coastal Massachusetts is supplied in Table 4.16.

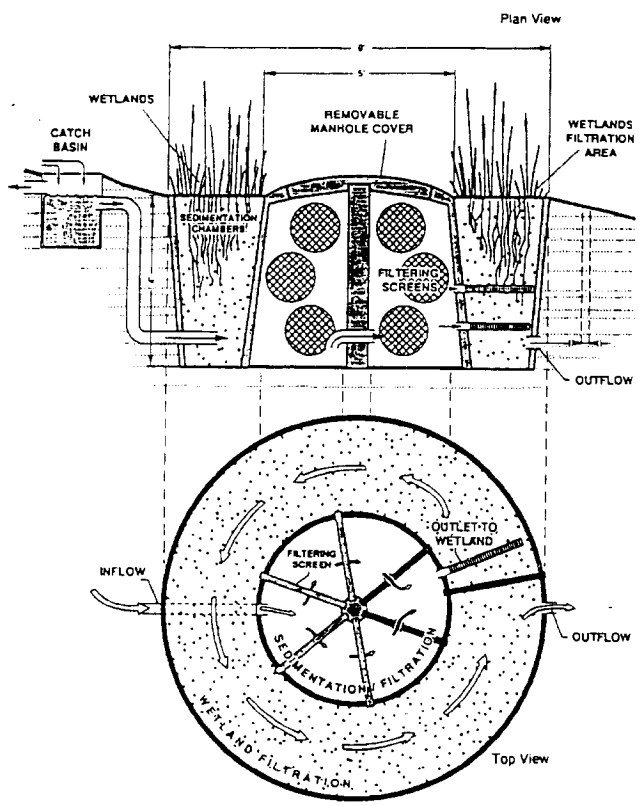
TABLE 4.15: POLLUTANT REMOVAL PERFORMANCE OF A ROCK WETLAND CELL FILTER SYSTEM IN FLORIDA (SOURCE: EGAN et al., 1995)

Parameter	*Mass Removal Rate (%)*
Total Suspended Solids-TSS	81
Total Dissolved Solids-TDS	8
Total Organic Carbon-TOC	38
Total Kjeldahl Nitrogen TKN	63
Nitrate-Nitrogen	75
Total Nitrogen	63
Ortho-phosphorus	14
Total Phosphorus	82
Cadmium	80
Chromium	38
Copper	21
Lead	73
Zinc	55
Fecal Coliforms	78

TABLE 4.16: POLLUTANT REMOVAL OF THE STORM TREAT SYSTEM (SOURCE: HORSLEY, 1995)

Parameter	*Stormwater Influent*	*Percentage Removed*
Fecal Coliform (no./100ml)	690	97
Total Suspended Solids	93	99
Chemical Oxygen Demand	95	82
Phosphorus (ug/l)	300	89
Dissolved N (ug/l)	1638	44
Total Petro HC (mg/l)	3.4	90
Lead (ug/l)	6.5	77
Chromium (ug/l)	60	98
Zinc (ug/l)	590	90

FIGURE 4.3: CROSS SECTION AND PLAN VIEW OF STORMTREAT SYSTEM



Reuter and his colleagues designed a simple submerged gravel filter in a high altitude, cold climate region in California. The filter treated the runoff produced from a 2.5 acre recreational area, most of which was fertilized ballfields (i.e., no impervious cover). The filter was a rather small 0.16 acres in size, composed of transplanted cattails that had not become fully established during the course of study. The bottom of the wetland was sealed with a liner, and filled with a three foot deep layer of fine gravel. Runoff was introduced into the gravel layer in a perforated pipe; outflow was collected by means of perforated pipe located in a standing well. Thus, runoff had to pass through the entire gravel filter before leaving the wetland. In general, the gravel layer was anaerobic (no oxygen), except for the top few inches. The bottom of the gravel layer was "innoculated" with muck from an adjacent wetland to introduce denitrifying bacteria into the system (Table 4.17).

TABLE 4.17: PERFORMANCE OF LAKE TAHOE GRAVEL-BASED STORMWATER WETLAND (SOURCE: REUTER et al., 1992)

Water Quality Parameter	*Mean Storm Removal (%)*
Suspended Sediment	80 to 88
Particulate Phosphorus	44 to 47
Soluble Reactive Phosphorus	-28 to -41
TKN	-3 to -58
NH4	-53 to -58
Nitrate	85 to 87
Soluble Iron	72 to 78

While the basic design of each gravel filter was somewhat different, each used a rock or gravel media, had standing water and had difficulty in getting wetland plants to colonize the media.

SUSPENDED SEDIMENT

All three gravel filter systems were able to remove at least 80% of the incoming suspended sediment concentrations, which may reflect the excellent settling environment with the gravel media (Wegelin, 1983). Removal of various forms of organic carbon ranged from 38 to 82%, which may be due to the export of algal detritus.

NITROGEN

The removal of organic forms of nitrogen in most gravel filters was generally high, ranging from 60 to 75%. Of even greater interest, each of the gravel filters were very effective in removing nitrate (or inorganic nitrogen), with Egan, Reuter and Horsley reporting 75%, 86% and 44% removal, respectively. As noted before, the high

nitrate removal rate is unusual among filtering systems, and may indicate that both nitrification and denitrification may be occurring in the aerobic and anaerobic environment present in the rock and gravel filter cells.

PHOSPHORUS

The gravel filters also exhibited a strong potential to remove total phosphorus, ranging from 44 to 89%. It should be noted that the two studies that actually measured soluble phosphorus removal recorded low or even negative rates, which may reflect "leakage" of internal biological production.

TRACE METALS

Egan reported variable removal of trace metals, with low to moderate removal for metals often found in soluble form (copper and chromium), and moderate to high removal for metals found primarily in particulate form (cadmium, lead and zinc). Horsley, on the other hand, reported removal rates ranging from 80 to 90% for chromium, lead and zinc.

BACTERIA AND PETROLEUM HYDROCARBONS

Monitoring of the StormTreat system indicated that this version of the gravel filter was able to remove an average of 97% of fecal coliform bacteria and 90% of incoming petroleum hydrocarbons.

RELATIVE IMPORTANCE OF GRAVEL MEDIA AND WETLAND PLANTS UPTAKE

The growth of algae and microbes among the gravel media was found to be the dominant removal pathway in the gravel filter, clearly outdistancing the effect of wetland plant uptake. Egan noted that unplanted rock filter cells performed better than any other planted cells, suggesting that wetland vegetation had no discernable influence on pollutant removal. He concluded that the rock surfaces themselves were important for pollutant removal, by creating a large substrate area for growth of epilithic algae and microbes, reducing flow rates, and providing more contact surfaces. The same basic conclusion was reached by Reuter et al. (1992) and Horsley (1995), since their gravel filter cells also never achieved extensive wetland plant coverage during the monitoring period.

In general, the pollutant removal performance of the packed bed filter was similar to those reported for sand and organic sand filters, with the notable exception of consistently higher rates for inorganic nitrogen.

4.3 COMPARATIVE POLLUTANT REMOVAL CAPABILITY

Several generalizations can be made about the overall performance of stormwater filtering systems. In general, they exhibit a high capability to remove suspended sediments, organic carbon and hydrocarbons, a moderate ability to remove total phosphorus and nitrogen (although low or negative with respect to soluble nutrient forms, and a moderate to high ability to remove trace metals pollutants (although, again, some designs are less effective at removing soluble forms). The one stormwater pollutant whose performance cannot easily be generalized is fecal coliform with some designs showing a high capability to remove bacteria, and others showing none. The average reported removal rates for the eleven stormwater filtering designs are compared in Table 3.5 in the last chapter.

How do the different stormwater filtering designs compare with respect to pollutant removal capability? Table 4.18 provides a general comparison of expected pollutant removal rates, based on monitoring data, theory and best professional judgement. As can be seen, most filtering designs have a high capability to remove sediment and hydrocarbons. Phosphorus removal rates range more widely, with the highest rates reported for gravel filters, dry swales and perimeter sand filters, and the lower rates for grass channels, wet swales and filter strips. Nitrogen removal typically ranges from 30 to 50%. Most filtering systems; however, have a zero or negative removal rate for soluble nitrate (with the exception of dry swales, wet swales and gravel filters). Most filtering systems have a high capability to remove bacteria, with the exception of open channel options such as drainage channels and grass channels. Metal removal rates are variable, but most designs appear capable of removing 50 to 75% of the total metal load delivered to them.

How does the performance of filtering systems, as a group, compare to other BMP systems, such as stormwater ponds, wetlands and infiltration systems? Table 4.19 presents a very generalized comparison of the comparative pollutant removal capability of these four groups of BMPs (important caveat: actual removal rates for a particular design within a BMP group, however, may be higher or lower than those shown in the Table, and are presented only for rough technology comparison).

When the four groups of BMP systems are compared, it is evident that there is not a great deal of difference in their capability to remove sediment, hydrocarbons or total phosphorus. Greater differences in pollutant removal are noted for nitrogen (especially nitrate), organic carbon, and trace metals. There is not enough data available to assess if their are any differences in bacteria removal among the four groups of BMPs. It should also be noted that the removal rates indicated for infiltration BMPs are projections only, since very few of these systems have actually been monitored. In summary, it appears that the removal capability of most BMP systems is similar for most pollutants of concern, *when they are designed and maintained properly and incoming pollutant levels are higher than the irreducible concentration.*

TABLE 4.18: ESTIMATED POLLUTANT REMOVAL CAPABILITY OF DIFFERENT STORMWATER FILTER SYSTEMS (AVERAGES OF REPORTED MONITORING DATA)

Filtering System	*Monitoring Data?*	*TSS*	*TP*	*TN*	*NO_3*	*Other Pollutants/Comments*
Surface Sand Filter	Yes, 6	85%	55%	35%	Neg	Bacteria: 40-80% Metals: 35-90%
Underground Sand Filter	No Data	Presumed to Comparable to Surface Sand Filter				
Perimeter Sand Filter	Yes, 3	80%	65%	45%	Neg	Hydrocarbons: 80%
Organic Sand Filter	Yes, 1	95%	40%	35%	Neg	Hydrocarbons: 90% Sol. P Negatives Metals: 85%+
Pocket Sand Filter	No Data	Presumed to be Comparable to Surface Sand Filter				
Drainage Channel	Yes, 10	30%	10%	Zero	Zero	Bacteria: Negative
Grass Channel = biofilter	Yes, 1	65%	25%	15%	Neg	Hydrocarbons: 65% Metals: 20-50% Bacteria: Negative
Dry Swale	Yes, 3	90%	65%	50%	80%	Metals: 80-90%
Wet Swale	Yes, 2	80%	20%	40%	50%	Metals: 40-70%
Bioretention	No Data	Presumed to be Comparable to Dry Swale				
Filter Strip	Yes, 1	70%	10%	30%	Zero	Metals: 40-50%
Gravel Filter	Yes, 2	80%	80%	65%	75%	Hydrocarbons: 85% Metals: 50-75%

TABLE 4.19
COMPARATIVE POLLUTANT REMOVAL CAPABILITY OF FOUR TYPES OF BMP SYSTEMS

Stormwater Pollutant	*Pond Systems**	*Wetland Systems*	*Infiltration Systems*	*Filtering Systems*
Suspended Sediment	80	75	90**	85
Organic Carbon	65	15	90**	50
Total Nitrogen	35	25	50**	35
Nitrate-N	60	60	50**	Negative
Total P	65	50	60**	60
Ortho-P	70	40	50**	50
Copper	50	30	60**	45
Lead	85	75	90**	85
Zinc	65	50	90**	75
Bacteria	1-2 Log	1-2 Log	1-2 Log**	2 Log
Hydrocarbons	80**	80**	?	85

Notes: * Does not include dry extended detention ponds
** Projected
The removal rates shown are for comparison purpose only
Actual removal for each system can vary widely depending on design
Sources: Current Assessment of Urban BMPs, Design of Stormwater Wetlands

4.4 DESIGN FACTORS TO ENHANCE FILTERING SYSTEM PERFORMANCE

In this section, practical design techniques are presented to consistently enhance the pollutant removal performance of stormwater filtering systems. These key design principles have been incorporated into the engineering methods presented in succeeding design chapters. Some general design principles that apply to all filtering systems include:

4.4A TYPE AND VOLUME OF PRETREATMENT

A pretreatment cell is not only needed to protect a filter from clogging, but also to temporarily store diverted runoff for subsequent treatment. Consequently, the

pretreatment volume is usually significantly greater for filtering systems than in other BMP systems. Where possible, some fraction of the pretreatment cell should be "wet" (i.e,. a permanent pool) to reduce incoming runoff velocities and reduce the potential for re-suspension of pollutants.

4.4B ADEQUATE CAPTURE VOLUME

It is important to capture and store a relatively large water quality volume (WQV) prior to treatment, since most filters are an off-line practices and will bypass some runoff during larger storm events (which is not treated). Based on consideration of rainfall/runoff statistics, monitoring data and pollutant removal pathways, it is practical to capture 90% of the average annual rainfall volume within or before the filter. In most regions of the Chesapeake Bay, this volume is equivalent to 1.0 inch of rainfall multiplied by the volumetric runoff coefficient (Rv) and the site area (acres). The capture volume for each filter can be temporarily stored in either the pretreatment cell or over the filter bed surface.

4.4C OFF-LINE FILTER DESIGN

Since filtering designs are intended to treat the water quality volume, they should be designed as off-line practices wherever possible. This usually involves constructing a flow-splitter or other device to divert the WQV into the filter bed. In cases where the filtering system must be designed on-line (e.g., grass channels, and dry and wet swales), it is important to ensure that the channel will not be subject to erosive runoff velocities during the 2 year design event (usually 4 or 5 feet per second).

4.4D SIZING OF FILTER BED

Each filtering system utilizes a slightly different area and depth for the filter bed. In most cases, the surface area of the filter bed is a direct function of the impervious area treated, and the depth of the filter bed ranges from one to two and a half feet (with the exception of bioretention areas, which are typically 4 feet deep). In most cases, the bulk of the filtration occurs with the top few inches of the filter media.

4.4E IMPROVED FILTER MEDIA

A common design approach has been to add a more organic media to the filter bed to enhance its removal capability. A series of organic media can be used for this purpose: peat, compost, organic soils to name a few. The limited data on organic media suggest that they may be superior in removing hotspot pollutants such as

hydrocarbons, metals, and organics (but also may be a net source of some nutrients due to leaching).

4.4F MULTIPLE POLLUTANT REMOVAL PATHWAYS

The key to improving the performance of any filter design is to maximize the value of settling, straining, infiltration, uptake or adsorption pathways within the system. Where possible, multiple pollutant removal pathways should be utilized to create a redundant treatment system.

4.4G PROMOTE PARTIAL EXFILTRATION

Filtering systems should be designed to exfiltrate runoff into the soil where insitu conditions allow, rather than collecting it in a pipe (groundwater contamination is not considered a risk). The partial exfiltration of runoff allows for additional pollutant removal by the soil layer. Infiltration of runoff can be a very important pollutant removal pathway, particularly for open channel designs.

4.4H IMPROVING NITROGEN REMOVAL

Many filtering systems have been found to have a poor ability to remove soluble nitrogen from urban runoff. If greater nitrogen removal is desired, it is important to promote greater denitrification within the filtering system. Usually, this is done by creating a wet cell or zone within the filter to maintain an anaerobic condition and a high organic matter content. This permanently saturated and anaerobic zone at the bottom of the filter bed creates favorable conditions for denitrifying bacteria, which might substantially improve the rate of nitrate removal. It is important to maintain an aerobic portion of the filter to avoid phosphorous leaching.

4.4I OPEN CHANNELS

To be effective, open channels should be explicitly designed to increase the volume of runoff that is retained or infiltrated within the channel, or at least lengthen the contact time through the channel during a storm. For best removal, open channels should be designed to retain/infiltrate the full water quality volume during a storm event.

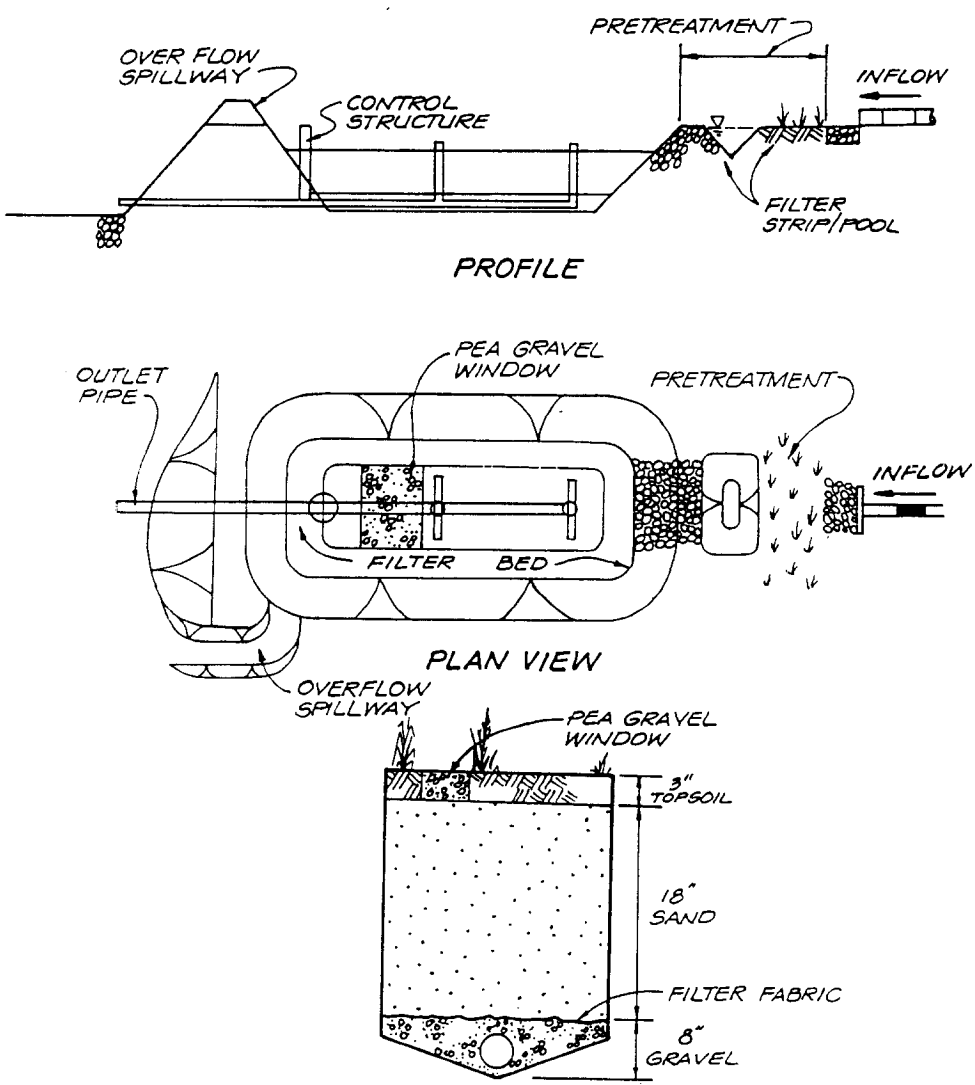
4.4J INTERNAL FILTER GEOMETRY

The hydraulics of each filter system should be carefully evaluated to ensure that incoming runoff does not "short-circuit" through either the pretreatment cell or the filter bed. In particular, a "drop-off" is often needed for many filtering systems to prevent an accumulation of sediment at the entry point to the filter. In addition, the designer should carefully evaluate whether the filter design meets minimum criteria for length, area or slope from the contributing drainage area or contact time.

Chapter

5

Key Design Elements of Stormwater Sand and Organic Media Filters



POCKET SAND FILTER

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CHAPTER 5
KEY DESIGN ELEMENTS:
SAND AND ORGANIC FILTERS

5.1 *INTRODUCTION*

The following three chapters follow the same general format adapted for the design of sand and organic filters, bioretention and open channel filters. Each practice has four major components: flow regulation, pretreatment, filter bed, and overflow. In addition, material specifications, construction considerations and maintenance elements are presented.

5.2 *ALTERNATIVE CONFIGURATIONS*

The five most common sand and organic filter alternatives, presented in Chapter 1, are reviewed again for clarity. They each were developed and adapted by various governments and engineers to serve different water quality treatment goals or to accommodate different physical constraints. Other alternative configurations may prove useful for different land use applications or climatic conditions and should be encouraged.

5.2A SURFACE SAND FILTER

The City of Austin, Texas first developed the sand filter technology for treatment of urban stormwater runoff in the early 1980's. The surface sand filter (or Austin sand filter, as it has often been called) is usually supported by a concrete shell, although earthen embankments are equally acceptable (Figure 5.1). The system is divided into a sedimentation chamber, for pretreatment to collect diverted runoff and settle out course sediments and a filter bed chamber, consisting of a flow distribution cell and the sand filter bed. The filter bed has an 18" - 24" sand layer which traps or strains pollutants before runoff is collected in an underdrain system (gravel and perforated pipe) and conveyed to the receiving stream, channel or pipe.

5.2B UNDERGROUND SAND FILTER

The underground sand filter (or District of Columbia sand filter) was developed for the intensely developed area within the inner city. This system is placed underground but maintains essentially the same components as the surface sand filter (Figure 5.2). The practice consists of a three chamber vault. A three feet deep wet sedimentation chamber is hydraulically connected by an underwater opening

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provide pretreatment by trapping grit and floating organic material. The second chamber contains an 18" - 24" sand filter bed and an underdrain system including inspection/cleanout wells. A layer of plastic filter cloth with a gravel layer can be placed on top of the sand bed to act as a pre-planned failure plane which can be replaced when the filter surface becomes clogged. The third chamber collects the flow from the underdrain system and directs flow to the downstream receiving drainage system (Truong, 1989).

FIGURE 5.1: SURFACE SAND FILTER

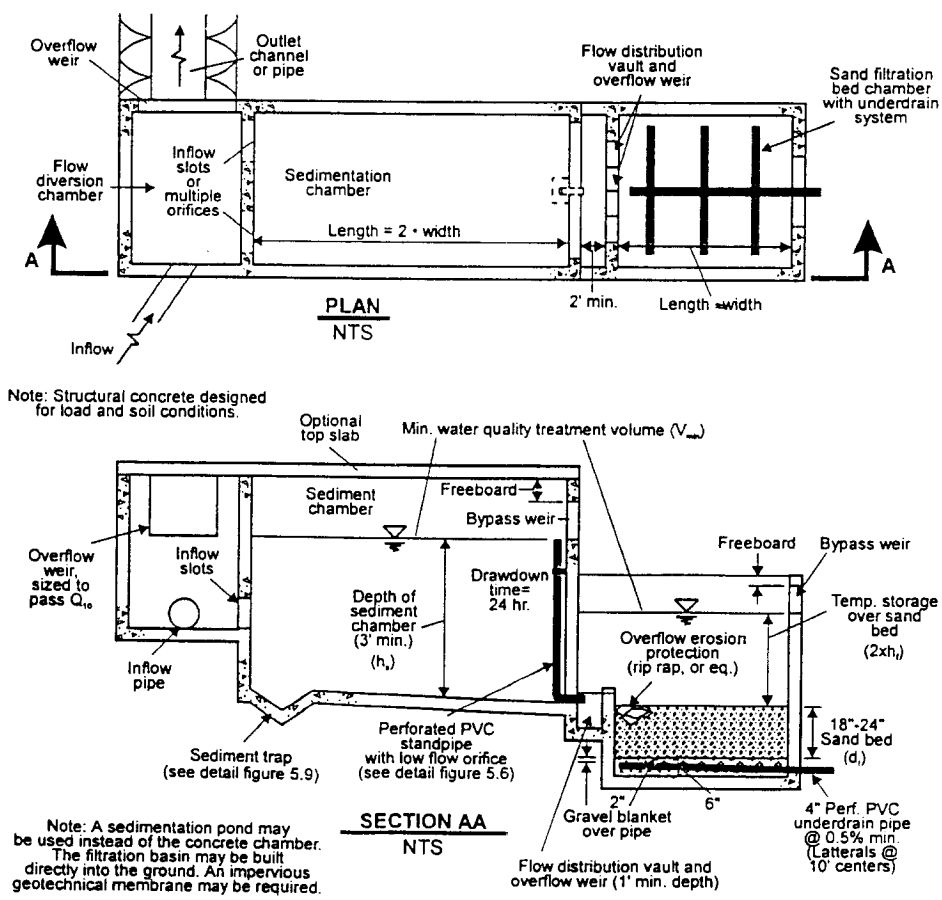
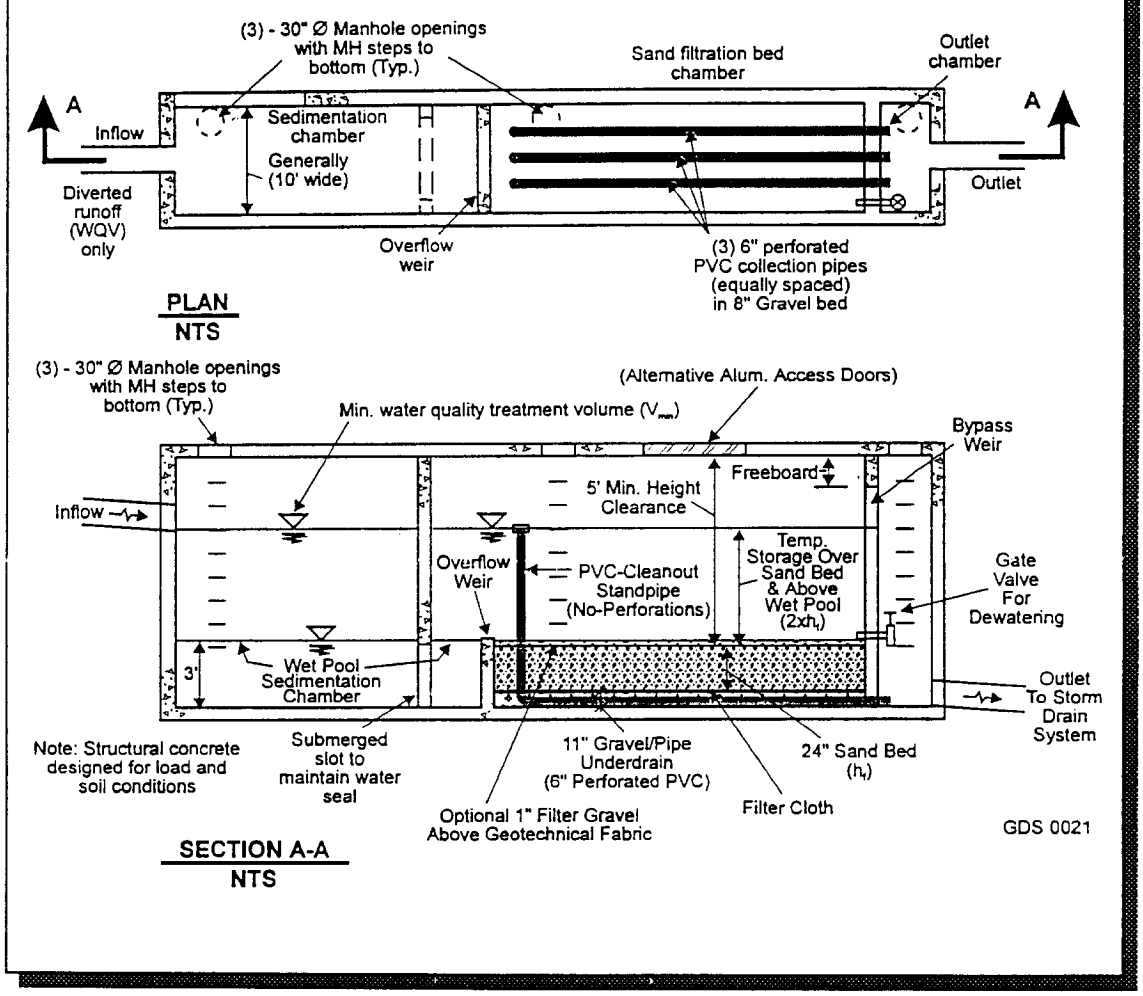


FIGURE 5.2: UNDERGROUND SAND FILTER

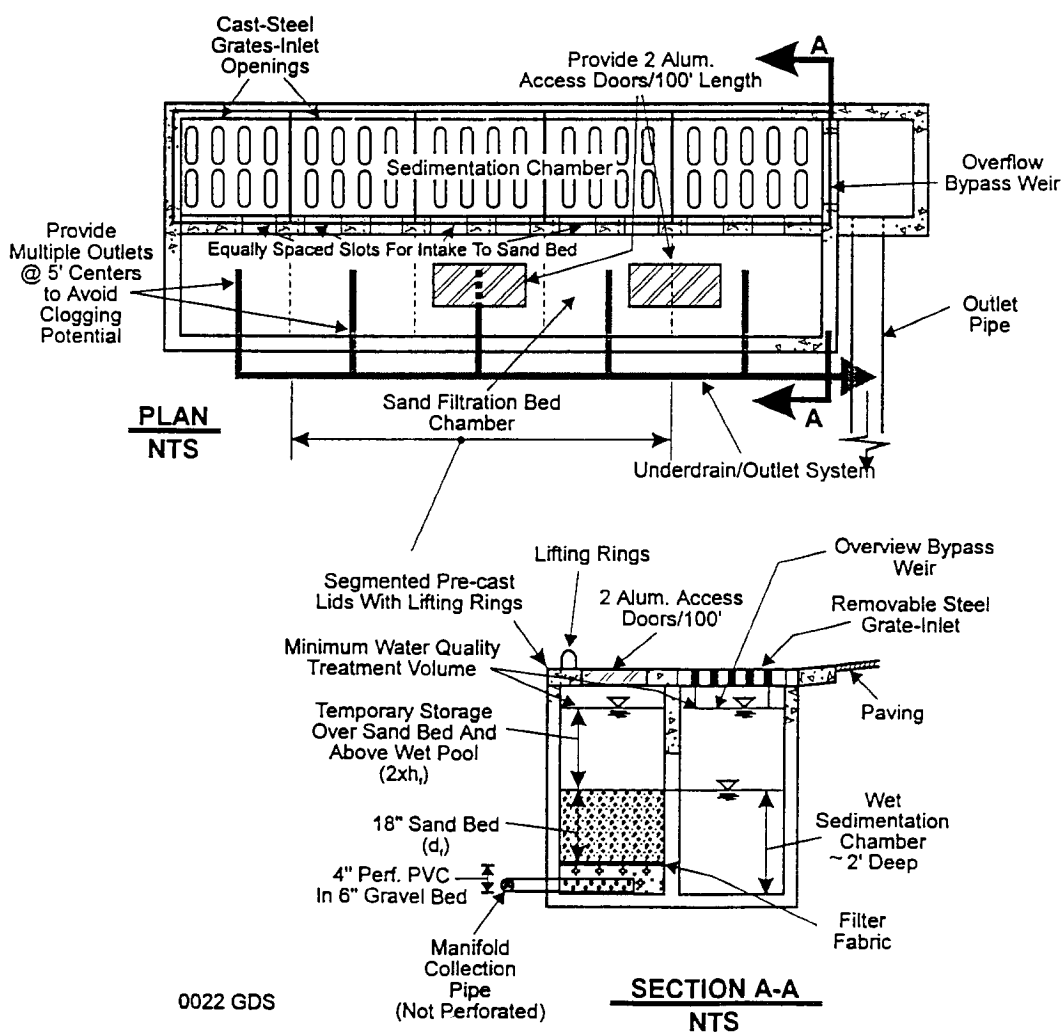


5.2c PERIMETER SAND FILTER

The perimeter sand filter was originally developed in Delaware by Earl Shaver and conceived as an on-line facility which treats all stormwater entering the system up to the overflow limit, originally set at the first one inch of runoff. The City of Alexandria, Virginia modified this system to incorporate a flow-splitter

to isolate and treat only the "water quality volume," (WQV). Figure 5.3 illustrates the modified Delaware sand filter. The system consists of a grated steel inlet to the sedimentation chamber, the filter bed itself, and an outlet chamber. The WQV flows into the filter bed via distribution slots or multiple orifices, while larger storm volumes bypass the filter chamber through an overflow weir. The filter bed chamber consists of an 18" sand bed over a gravel/perforated pipe underdrain system with several outlets which discharges into a manifold pipe collection system.

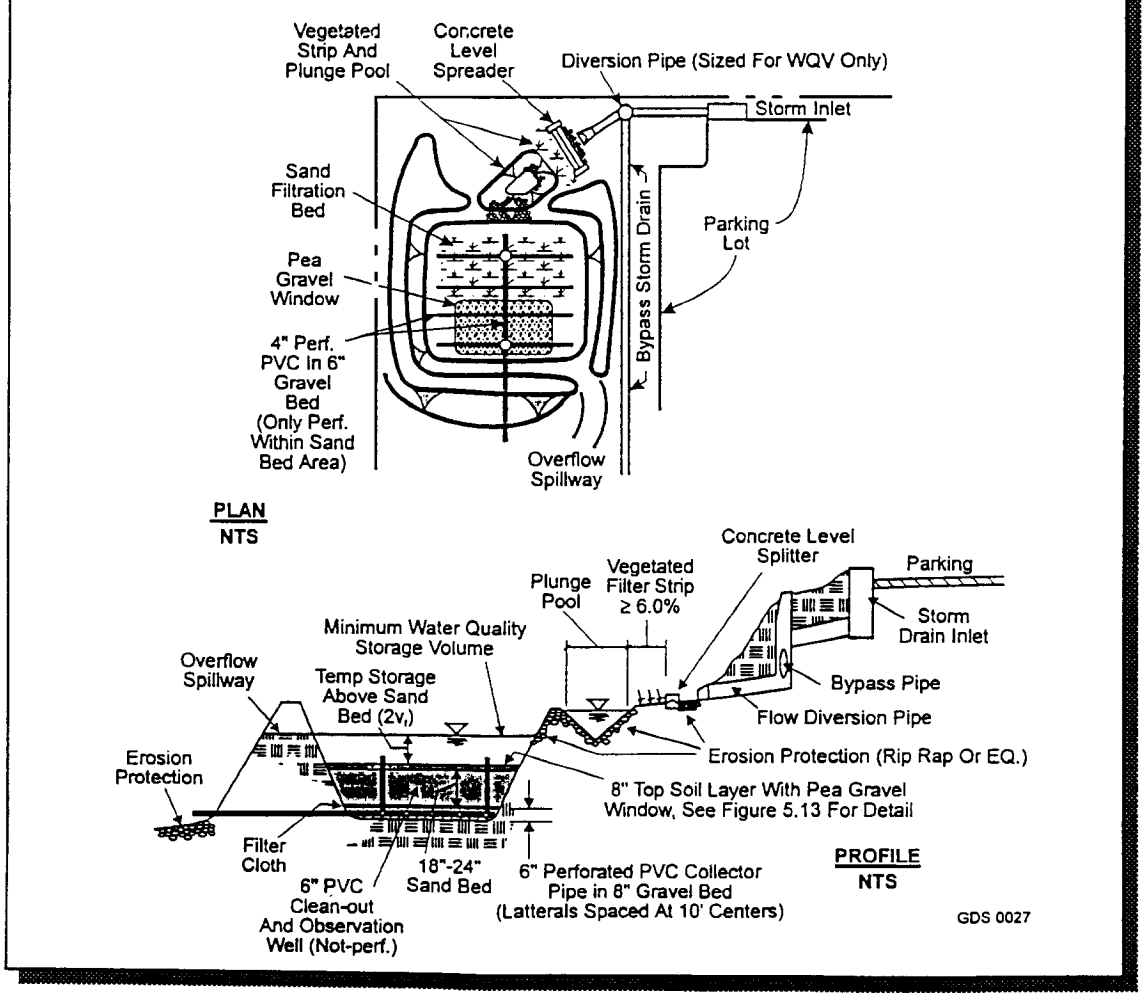
FIGURE 5.3: PERIMETER SAND FILTER



5.2D POCKET SAND FILTER

The pocket sand filter system is intended to provide an inexpensive solution for utilizing sand filter technology for those small sites where anticipated sediment loads do not warrant a sedimentation chamber and can suffice with vegetative pretreatment practices. The pocket sand filter (Figure 5.4) consists of a flow splitter inlet structure to capture the WQV, a vegetative filter strip or suitable alternative (such as a small stilling basin at a storm drain pipe outfall) and an above ground sand filter bed (18"-24") over a gravel underdrain system. The filter bed chamber may require an impermeable liner for areas where groundwater contamination is a critical concern. The pocket sand filter may also be constructed on-line for very small drainage areas (say, less than one acre). In these cases, a conventional inlet and discharge structure is necessary to accommodate both the WQV and larger storms.

FIGURE 5.4: POCKET SAND FILTER



5.2E ORGANIC FILTER MEDIA

The use of an organic media within a filtering system may offer increased pollutant removal efficiencies over sand filters alone (particularly with respect to nutrients). At least two principal types of organic media have been utilized recently for the treatment of urban runoff. These are peat (partially decomposed organic material of geological origin) and leaf compost.

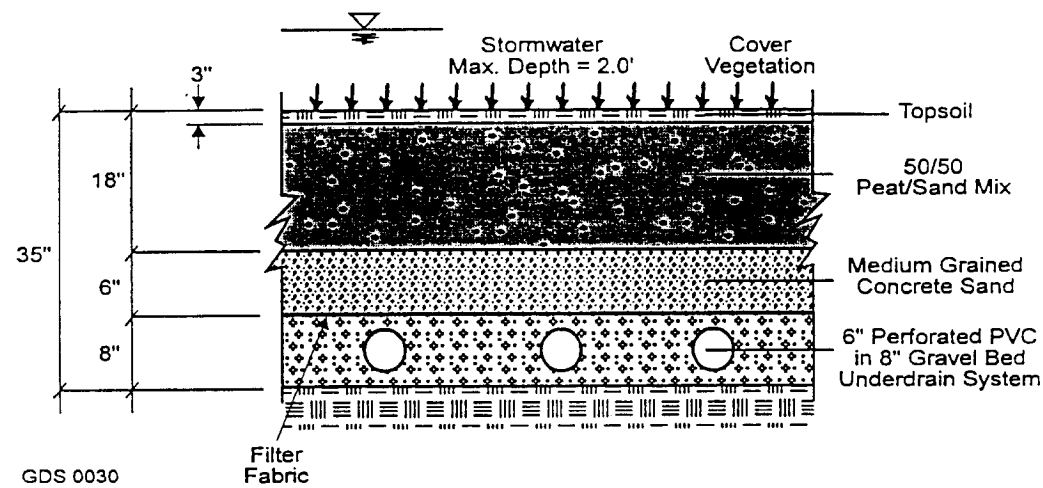
Peat has been utilized in conjunction with sand as an alternative wastewater treatment system for several years. Recently there have been attempts to adapt this practice for stormwater applications. John Galli, of the Metropolitan Washington Council of Governments, prepared an analysis paper of peat-sand practices for stormwater applications (Galli, 1990). The City of Alexandria has incorporated the work of Galli and others into a design criteria (City of Alexandria, 1992).

A proprietary leaf compost system has been developed by CSF Treatment Systems, Inc. of Portland Oregon for treating stormwater runoff from smaller drainage areas. Approximately 30 compost systems have been installed in the Pacific Northwest over the past several years.

PEAT-SAND FILTER SYSTEM

The peat-sand filter system was originally developed in the Pacific Northwest and consists of a fabricated soil filtration bed which combines the benefits of peat with those of a grass cover crop and a sand underlayer. The type of peat utilized in the filtration bed is extremely important. A fibric peat, where the undecomposed fibrous organic matter is easily identifiable is the preferred type. A hemic peat, where more material is decomposed, may also be utilized. Under no circumstances should a sapric peat, made up of mostly decomposed matter, be used. Figure 5.5 illustrates a typical peat-sand filtration bed cross section. The surface sand filter application can be modified to incorporate the peat-sand filtration bed.

FIGURE 5.5: PEAT-SAND CROSS SECTION

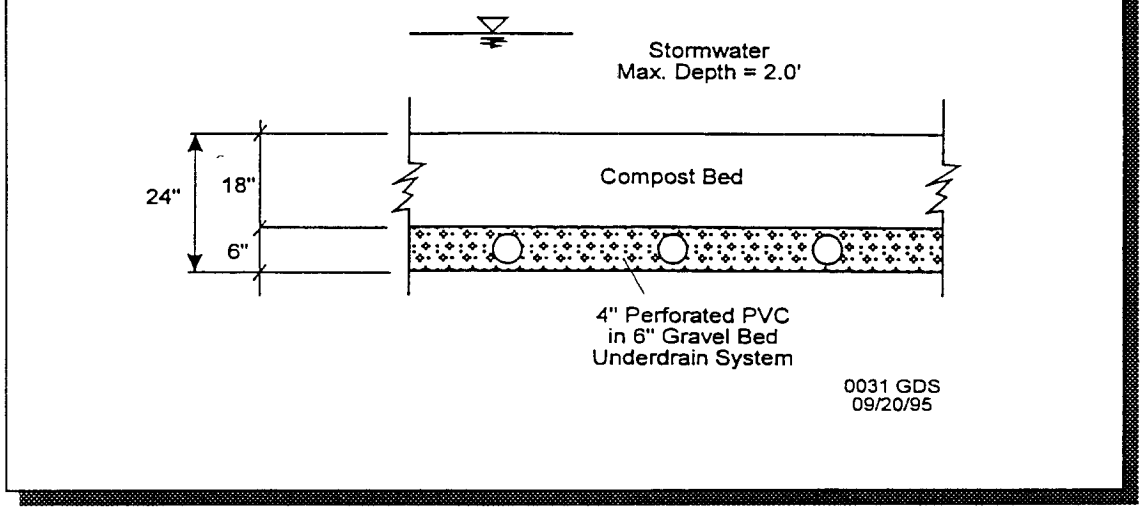


COMPOST FILTER SYSTEM

The compost filter system consists of a fabricated leaf compost filtration bed overlying a gravel/pipe underdrain system. The key to the system is through the proper selection of compost. The compost should be mature and humic (the organic material is no longer rapidly degrading), have low contaminant levels, have a high permeability, and be locally available at a reasonable cost. A leaf compost medium, as opposed to a yard waste compost mixture is necessary. A high quality leaf compost is prepared by ensuring weekly turning which promotes good size reduction, aeration and rapid maturation. Some road gravel is often included in the compost which helps afford good flow permeability. Recently, a pelletized compost is being employed to maintain higher filtration rates (CSF, 1996). Figure 5.6 illustrates a typical compost filtration bed cross section. The surface sand filter application can be adapted for the compost filter media.

CSF Treatment Systems, Inc. (1994) has a proprietary system which incorporates a design size based on a compost bed surface area requirement of 200 ft^2/cfs, a filter media thickness of 18 inches, a forebay of unspecified size, a gravel/pipe underdrain system, and a discharge structure.

FIGURE 5.6: COMPOST FILTER CROSS SECTION



5.3 FLOW REGULATION

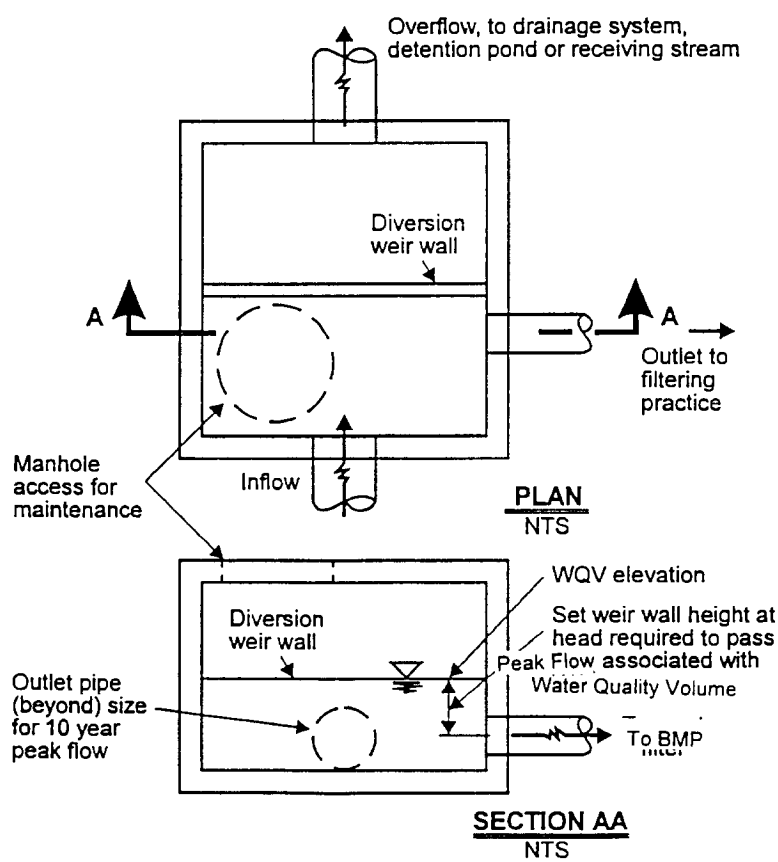
Since sand filters are designed to provide treatment for the "water quality volume" (WQV) only, they should be located off-line from the primary conveyance/detention system. Sand filters should be located where they can intercept as much of the site impervious area as possible and where discharge to the primary conveyance system is feasible.

Offline designs are recommended for sand filter systems to avoid mixing with larger storm events which are likely to resuspend settled solids within the sedimentation chamber, scour the filter bed, or otherwise compromise the pollutant removal effectiveness of these facilities.

The design objective is to capture and divert the water quality volume (WQV) to the sand filter and "bypass" larger storms to the downstream storm drainage system or receiving water. WQV is computed based on the methods identified in Chapter 2. In most Chesapeake Bay drainage jurisdictions, the enclosed conveyance systems are sized for the 10 year storm event. Open channel systems may be sized for larger events. A flow diversion structure must be able to accommodate these larger flows as well as the water quality storm.

Two methods for diverting the WQV include: Computing a peak discharge (Q_p) for the "water quality storm" and utilizing an isolation/diversion structure upstream and within the drainage network or, incorporating the isolation/diversion structure within the treatment practice itself. Figure 5.7 illustrates an application of the first method. See Figure 5.1 for application of the second method.

FIGURE 5.7: ISOLATION/DIVERSION STRUCTURE



The preferred method for accomplishing a diversion is within the treatment practice itself, where the overflow (or bypass) weir elevation is set equal to the design WQV elevation within the adjacent practice. This method ensures larger inflows will overflow the bypass weir, thus minimizing mixing within the BMP. It is also a more reliable capture technique, than reliance on a computed peak rate of discharge (Qp) to size the diversion structure.

It is still necessary to compute the Qp to size the intake slots or openings. The openings directing runoff to the treatment practice should be slightly oversized to ensure that the entire WQV is treated. The design example at the end of this chapter illustrates the methodology for doing this.

In many cases, however, it is not possible to maintain the necessary geometry and elevations to locate the isolation/diversion structure within the treatment practice itself. Therefore, an alternative technique for isolation/diversion within the drainage network should be utilized. The methodology for doing this is described in Table 5.1:

TABLE 5.1
DESIGN PROCEDURE FOR DIVERSION TECHNIQUE WITH THE DRAINAGE NETWORK

1.	Peak Discharge (Qp) for WQV is computed based on the methods presented in Chapter 2.
2.	Qp for the "bypass storm" is computed (most jurisdictions utilize the 10 year frequency storm). Utilize the Rational Formula or SCS TR-55.
3.	Size diversion slots/openings or pipe utilizing the orifice equation: $Q=CA(2gh)^{1/2}$
4.	Size overflow weir for "bypass storm" using the Weir Equation: $Q = C_W L H^{3/2}$, size the outfall pipe, if provided, using the orifice equation (to check inlet condition flow capacity) and Manning's equation to check friction losses.

5.4 PRETREATMENT

Pretreatment is necessary for stormwater filters to remove excessive sediment which contributes to premature failure of the practices. Pretreatment may be in the form of sedimentation basins, vegetative filter strips, grass swales, storm drain structures with sumps, or water quality (oil/grit separator) inlets.

5.4A SEDIMENTATION BASINS

Sedimentation basins, also called pre-settling basins, are the preferred method of pretreatment for stormwater filters because the basins are constructed in conjunction with the filter bed and maintenance requirements are relegated to one location. In addition, the performance and sizing criteria for sedimentation basins are reasonably well established. Sedimentation basins can also be constructed underground in high density areas where space is limited.

The water quality volume, computed in Chapter 2, is used as the basis for sizing the pretreatment chamber for all types of stormwater filters, except the "pocket system." According to an extensive literature review conducted by the City of Austin, TX, removal of discrete particles by gravity settling is primarily a function of surface loading (the rate of outflow divided by the basin surface area) and is independent of basin depth (Washington State Department of Ecology, 1992). However, a minimum basin depth of 3 feet is recommended to minimize particle resuspension and turbulence effects. Therefore, surface area is the primary design parameter for sedimentation affecting removal efficiency (E). E is also a function of particle size distribution. Silt sized particles are used as the target particle size for sedimentation basin design (i.e., ≈ 20 microns).

For sites with imperviousness ≥ 75%, which have a higher percentage of coarse grained sediments (Shaver and Baldwin, 1991), the target capture partical is approximately 40 microns.

The following equation is used to size pretreatment settling basin surface area. It was derived by the Washington State Department of Ecology from the Camp-Hazen equation (Washington State Department of Ecology, 1992 and Chen, 1975).

$A_s = -(Q_o/w) * Ln(1-E)$ where: **Equation 5.1**

A_s = Sedimentation basin surface area (ft^2)

E = Trap efficiency; which is the target removal efficiency of suspended solids (set

equal to 90%)

w = Particle settling velocity; for target particle size (silt) use settling velocity = 0.0004 ft/sec (0.0033 ft/sec for I $\geq$ 75%, where I is percentage impervious area

Q_o = rate of outflow from the basin; which is equal to the water quality volume (WQV) divided by the detention time (t_d); use 24 hours.

$Q_o = WQV/t_d$ therefore:

$A_s = -WQV / [(24\ hr)(3600\ sec/hr)(0.0004\ ft/sec)] * Ln(0.1)$

$\mathbf{A_s = \underline{0.066 * (WQV)\ ft^2}}$ **Equation 5.2**

$\mathbf{A_s = \underline{0.0081\ (WQV)\ ft^2}}$ for I $\geq$ 75% **Equation 5.2.1**

As discussed in Chapter 1, the WQV is used as a basis for sizing all filtering practices. However, for sand and organic media filters, where pervious areas are intentionally limited, the runoff for the WQV can be a sizable quantity and complete storage of the WQV is often not feasible or is cost prohibitive. Therefore, although the WQV is used to size minimum surface areas for both the sedimentation and filter bed chambers, **a volume of three-quarters of the WQV is maintained as the minimum storage volume required.**

$\mathbf{V_{min} = \frac{3}{4} * WQV}$ **Equation 5.3**

Storing three-quarters of the WQV versus 100% of WQV is justified because the sedimentation chamber is continually draining into the filter bed during the course of a storm event. Only short duration, high intensity storms are likely to exceed the three quarters WQV threshold.

The length to width ratio of the basin should be 2:1 or greater. Inlet and outlet structures should be located at extreme ends of the basin. Baffles may be used to mitigate short-circuiting and/or dead storage problems. The basin bottom shall have a minimum depth of 3 feet to minimize resuspension and turbulence. The basin bottom shall be nearly level to facilitate sedimentation. The design elements for pretreatment which are specific to the different design variations are presented in Table 5.2.

TABLE 5.2: PRETREATMENT COMPONENTS FOR FOUR DESIGN VARIATIONS

Surface Sand Filter:

- Dry detention basin.
- Minimum volume = ¾ * WQV: split between volume within filter bed (voids), volume above filter bed, and volume within pretreatment chamber.
- Perforated standpipe with orifice sized to release volume (within sedimentation basin) over 24 hour duration (Figure 5.8). Note: The size and number of perforations depends on the release rate needed to achieve 24 hour detention.
- Overflow weir within the sedimentation chamber is set at design treatment volume, sized to pass 2/3 of WQV peak flow. Overflow weir within sand bed chamber set at design treatment volume, sized to pass 1/3 of WQV peak flow. This ensures at least partial treatment for flows exceeding ¾ * WQV.
- Permanent sediment trap: Since the sedimentation basin is dry, a permanent sediment trap is recommended. This consists of a small storage area to trap incoming sediment and remove this from the basin flow regime. It is recommended that the sediment trap volume be equal to ten (10) percent of the sedimentation basin volume. Water collected in the trap is conveyed directly to the flow distribution vault (Figure 5.9).

Underground Sand Filter:

- Wet retention basin.
- Wet volume (Vw) = A_s * depth (3' deep, minimum permanent pool storage).
- Total minimum volume = ¾ * WQV: Split between volume within filter bed (voids), wet volume within sedimentation chamber, volume above wet volume, and volume above sand bed.
- Overflow weir elevation (in filter chamber) set at design treatment volume, sized to pass 2/3 of WQV peak flow.

Perimeter Sand Filter:

- Wet retention basin.
- Wet volume (Vw) = A_s * depth (2' minimum depth permanent pool storage).
- Total minimum volume = ¾ * WQV: Split between volume within filter bed (voids), wet volume within sedimentation chamber, volume above wet volume, and volume above sand bed.
- Elevation of overflow weir to outlet chamber set at top of dry storage elevation (¾ * WQV), sized to pass 100% of incoming 10 year design flow.

Organic Filter Media

- The pretreatment technique for organic media is the same as with the surface sand filter, or it can incorporate a wet retention component as well (as with the perimeter and underground sand filter).

FIGURE 5.8: PERFORATED STANDPIPE WITH DRY SEDIMENTATION BASIN

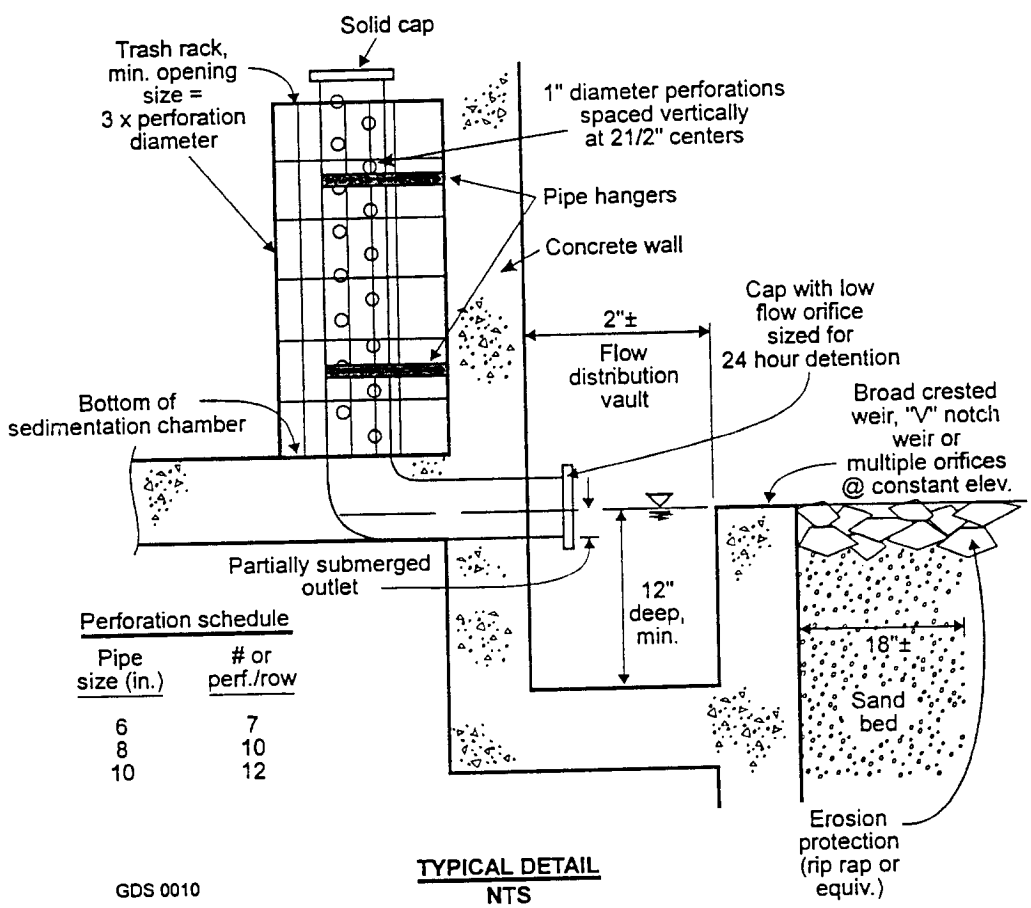
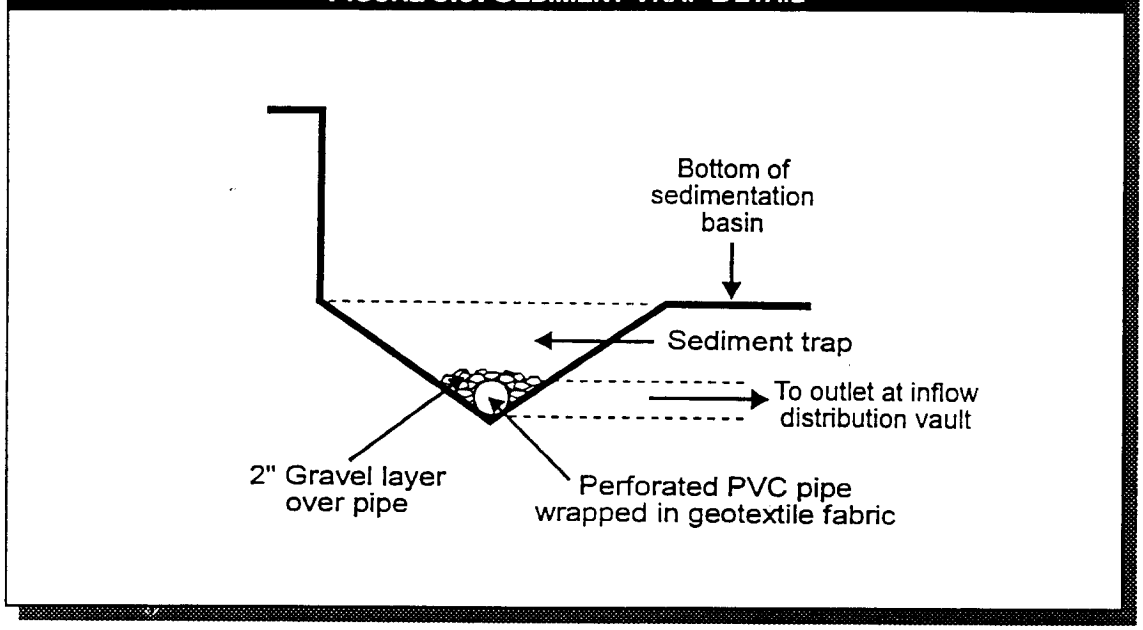


FIGURE 5.9: SEDIMENT TRAP DETAIL



5.4B VEGETATIVE PRACTICES

Vegetative filter strips and grass swales may be useful pretreatment methods where adequate space and conditions permit. Design parameters for these practices are reviewed in detail in Chapter 7. If the practice is being used for pretreatment for sand or organic filters, the design length or volume can be reduced to **10% of the required "stand-alone" design length.** The principal pretreatment components for the pocket sand filter are presented in Table 5.3.

TABLE 5.3
PRINCIPAL PRETREATMENT COMPONENTS FOR POCKET SAND FILTER

▸ Concrete level spreader for facilitating sheet flow
▸ Vegetated filter strip for partial pre-treatment (5' minimum)
▸ Plunge pool stilling basin for partial pre-treatment

5.4C STORM DRAINAGE SUMP INLETS

Storm drain structures with sumps can provided some reduction in incoming sediment loads but require frequent cleaning to avoid resuspension of solids. If a jurisdiction does not have a strenuous maintenance program with frequent storm drain system cleaning, this method is not encouraged.

5.4D WATER QUALITY INLETS

Water quality inlets have been shown to be a marginal method for removing particulate matter according to a study by the Metropolitan Washington Council of Governments (Schueler and Shepp,1993), and are therefore not recommended for sand filter pretreatment.

5.5 FILTER MEDIA

5.5A GENERAL SIZING GUIDANCE

The principles of Darcy's Law are used for sizing the sand filter bed area as derived by the City of Austin, TX, Environmental and Conservation Services Department (City of Austin, TX 1988).

The primary design parameter for filtration basins is surface area. The necessary surface area is a function of the permeability of the filter medium, the bed depth, the hydraulic head (height of water above the bed), and sediment loading. The following equation can be used to size all types of filter media presented in this manual.

$A_f = WQV * (d_f)/[k*(h_f + d_f)(t_f)]$ where: **Equation 5.4**

A_f = Surface area of the sand filter bed (ft^2)
WQV = Water quality treatment volume (ft^3)
d_f = Sand filter bed depth (ft)
k = Coefficient of permeability for sand bed (ft/day)
h_f = Average height of water above the sand bed (ft); $h_f = ½*h_{max}$
t_f= Time required for the Water Quality Treatment Volume (WQV) to filter through the sand bed

- WQV is computed using the procedures outlined in Chapter 2.
- d_f can vary depending on the site conditions but should not be more 24" (18" is the standard).
- h_f will also vary depending on the site conditions, but should not exceed 6 feet.
- A value of 40 hours is recommended for the filter bed draw-down time (t_f).

K VALUES FOR SAND FILTERS

k values for sand were computed by the City of Austin staff based on field observation and actual performance of previously installed sand filters. The values ranged form approximately 0.5 to 2.7 ft/day, with an average value of 1.5 ft/day. These values are substantially lower than those quoted in textbooks (Hwang, 1981) but allow for clogging associated with accumulated sediments. With an appropriately sized sedimentation basin (as described above), a value of **k = 3.5 ft/day** is recommended (City of Austin, TX, 1988).

K VALUES FOR PEAT-SAND FILTERS

A composite coefficient of permeability is used based on the 50/50 mixture thickness of the different media.

$k = k_1 + k_2/2$

For peat, k can range from as high as 110 ft/day to as low as .02 ft/day depending on whether the peat is fibric, hemic, or sapric (Galli, 1990). Galli (1990) and Bell (1993) use a 2 ft/day coefficient of permeability for surface area sizing considerations (for a mixture of fibric and hemic peat).

Based on the broad range of peat permeability and its superior water holding capacity, a **coefficient of permeability of 2 ft/day** is recommended for design.

Using a k of 2 ft/day for peat, 3.5 ft/day for sand, and the typical section illustrated in Figure 5.5, an average coefficient of permeability of 2.75 ft/day is recommended.

K VALUES FOR COMPOST FILTERS

CSF Treatment Systems, Inc. (1994) recommends using permeability of 2.25 gpm/ft^2 for compost, based on laboratory tests. This translates to a permeability coefficient of 433 ft/day. Stewart (1992), presented data that showed that while the initial permeability of compost was always high, it tapered off after approximately 24 hours to generally 20% of the initial rate. Based on these results, the initial permeability rate should be reduced by 80%

The coefficient of permeability should reflect design conditions prevalent locally. In the Chesapeake Bay watershed, where rainfall characteristics differ substantially from the Pacific Northwest, it is important to capture and treat the high intensity rainfall events which form a significant portion of the annual runoff. In addition, since surface clogging does occur on filtering practices, the design permeability rate should reflect a percentage of the laboratory results after several hours of rainfall

have occurred. A rate equivalent to 10% of the diminished rate should be used. Therefore the recommended design k value for compost should be as follows:

433 ft/day * 0.20 * 0.10 = 8.66 ft/day, use **k = 8.7 ft/day**

TABLE 5.4: COEFFICIENT OF PERMEABILITY VALUES FOR STORMWATER FILTERING PRACTICES

Filter Media	*Coefficient of Permeability (k, ft/day)*
Sand	3.5
Peat/Sand	2.75
Compost	8.7

5.5B SIZING PROCEDURES FOR DESIGN VARIATIONS

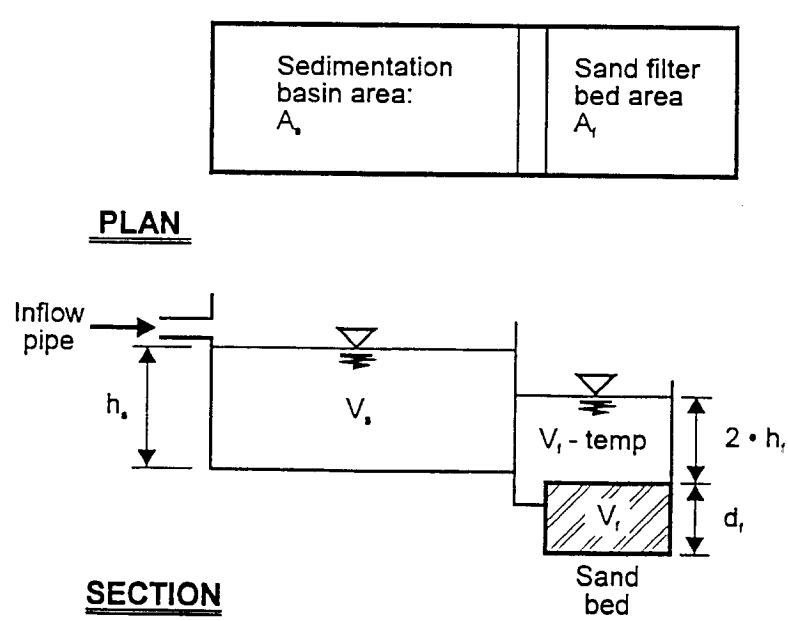
Sizing sand and organic filters is in most cases a straightforward process. Listed below is the sizing procedure for each design variation. Items 1 through 6 are the same for all variations, which include computing the water quality volume and flow regulation to the facility. Identified separately is the process for each design variation.

1. Compute the Water Quality Treatment Volume (WQV).
2. Calculate the peak discharge (Q_p) utilizing the 90% Rule (from Chapter 2), 1.0" rainfall for Chesapeake Bay Watershed.
3. Size the flow diversion structure to divert the WQV to the sand filter.
4. Using Darcy's Law, size the sand filter bed surface area (A_f).
5. Using the Camp-Hazen equation, size the sedimentation basin surface area (A_s)(except pocket sand filter).
6. Compute the required minimum storage within the practice (V_{min} = ¾ * WQV)

SURFACE SAND FILTER (FIGURE 5.10)

- Compute the water volume within the filter bed (V_f) = A_f * depth of bed & gravel (d_f) * porosity (n) (use n = 0.4 for sand/gravel/perforated pipe).
- Compute the temporary storage volume above the filter bed ($V_{f\text{-temp}}$)= 2 * h_f * A_f.
- Compute the remaining volume required for the dry settling basin (V_s) = V_{min} - (V_f + $V_{f\text{-temp}}$). Note: V_s should be approximately to 50% of V_{min}. If not, decrease h_f, and recompute.
- Compute height (h_s) in settling basin chamber = V_s/A_s.
- Check to make sure $h_s > 2 * h_f$, and $h_s \geq 3'$, if not adjust h_f and repeat procedure.

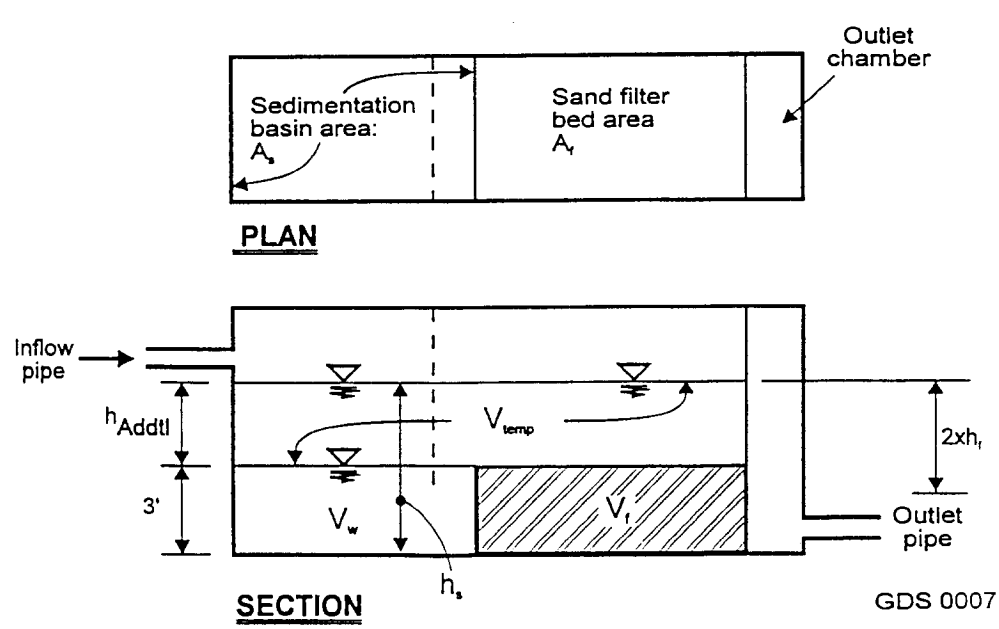
FIGURE 5.10: SCHEMATIC-SURFACE SAND FILTER



UNDERGROUND SAND FILTER (FIGURE 5.11)

- Compute the water volume within the filter bed (V_f) = A_f * depth of bed & gravel (d_f) * n.
- Compute the minimum wet pool volume in the settling basin (V_w) = A_s * 3' minimum.
- Compute the temporary storage volume required within both chambers (V_{temp}) = V_{min} - (V_f +V_w).
- Compute the total surface area of both chambers (A_f + A_s).
- Compute additional temporary storage height (h_{addtl}) = V_{temp} /(A_f + A_s).
- Check to make sure $h_{addtl} \geq 2 * h_f$ (from Darcy's Law), if not, decrease h_f and recompute.

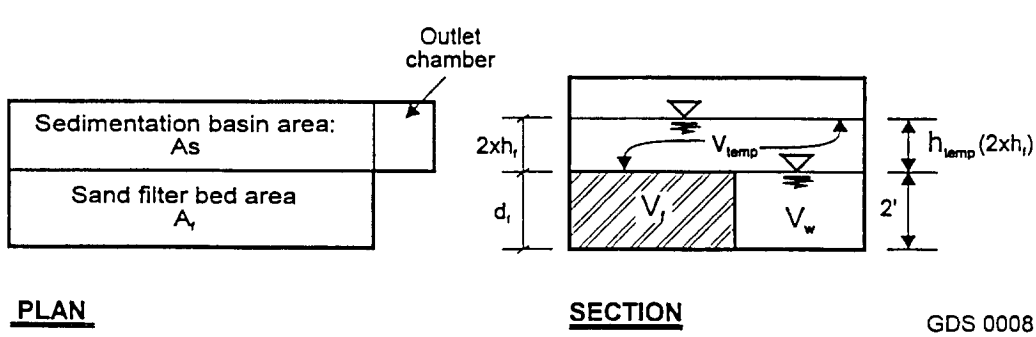
FIGURE 5.11: UNDERGROUND SAND FILTER



PERIMETER SAND FILTER (FIGURE 5.12)

- Compute the water volume within the filter bed (V_f) = A_f * depth of bed & gravel (d_f) * n.
- Compute the minimum wet pool volume in the settling basin (V_w) = A_s * 2' minimum.
- Compute temporary storage volume required (V_{temp}) = V_{min} - (V_f +V_w).
- Compute the total surface area of both chambers (A_f + A_s).
- Compute temporary storage height (h_{temp}) = V_{temp}/(A_f + A_s).
- Check to make sure $h_{temp} \geq 2 * h_f$ (from Darcy's eq.), if not, decrease h_f and recompute.

FIGURE 5.12: PERIMETER SAND FILTER



POCKET SAND FILTER

- Compute the water volume within the filter bed (V_f) = A_f * depth of bed & gravel (d_f) * n.
- Compute the temporary storage volume required (V_{temp}) = V_{min} - V_f.
- Compute the temporary storage height (h_{temp}) = V_{temp}/A_{avg}, where A_{avg} is the average area of the pocket sand filter.
- Set overflow spillway elevation = h_{temp}.

ORGANIC FILTER MEDIA

- Compute the water volume within the filter bed (V_f) = A_f * depth of bed & gravel (d_f) * porosity (n) (porosity of organic layer, plus gravel/pipe system will vary depending on the practice filter medium, from approximately 0.33 to 0.4).
- Compute the temporary storage volume above the filter bed ($V_{f\text{-}temp}$) = 2 * h_f * A_f.
- Compute the remaining volume required for the settling basin (V_s) = V_{min} - (V_f + $V_{f\text{-}temp}$). Note: V_s should be approximately to 50% of V_{min}. If not, decrease h_f, and recompute.
- Compute height (h_s) in settling basin chamber = V_s/A_s.
- Check to make sure $h_s > 2*h_f$, if not adjust h_f and repeat procedure.

5.6 OVERFLOW

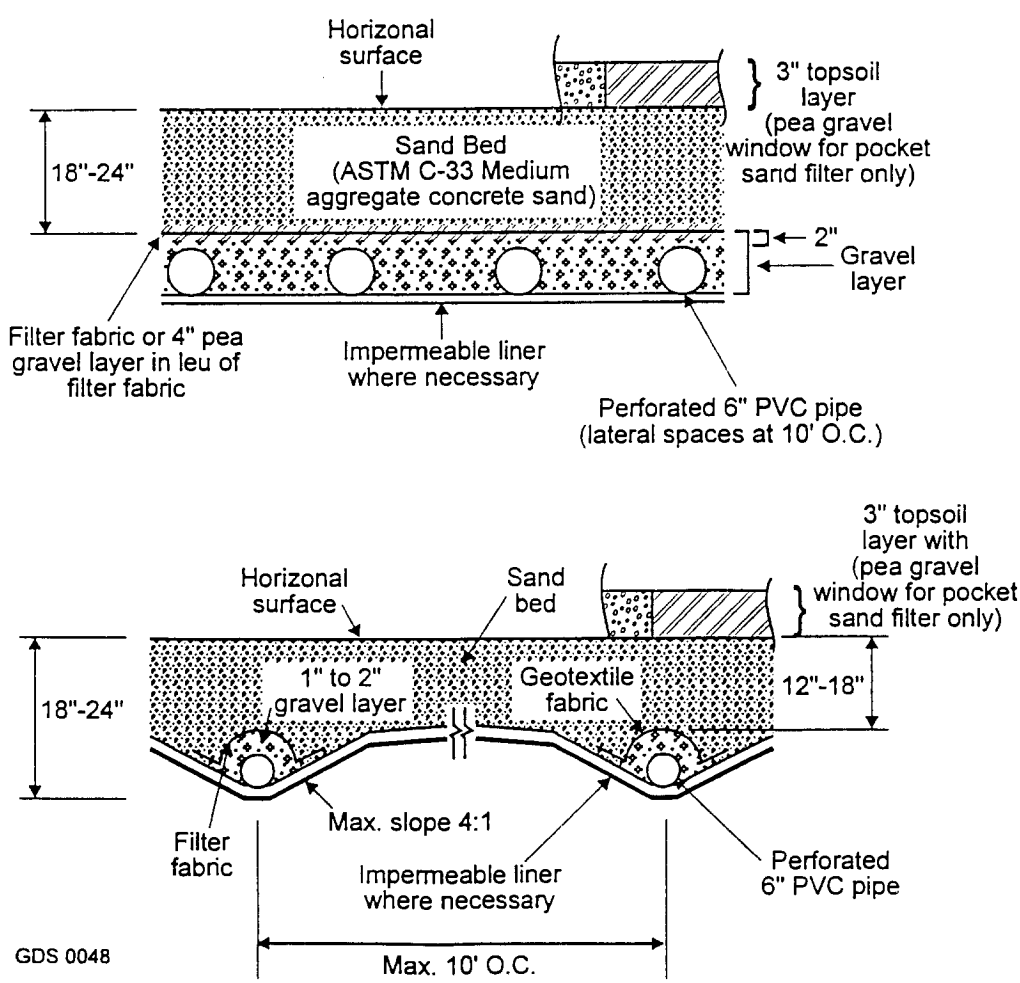
The overflow elements of the filter chamber consist of a flow distribution vault, a sand or organic media filter bed, underdrain piping, a basin liner (whenever necessary to prevent groundwater contamination), and a high flow overflow structure. In some applications the filter bed will have a cover of either vegetation, gravel or a synthetic geotextile-matrix matting.

The flow distribution vault should be designed to spread the flow uniformly across the surface of the filter bed. V-notch weirs, a level broad crested weir or multiple orifice openings are alternatives. Flow should be dispersed in a non-scouring way. The height of the inlet structure should be equal to the filter bed elevation. Rip rap or other suitable erosion protection should be installed immediately below the inlet structure discharge location where necessary.

The filter bed profile (Figure 5.5, 5.6, and 5.13) generally consists of a cover vegetation planted in a 3" topsoil layer above an 18" layer of 0.02"-0.04" diameter, clean concrete sand or organic media, and over a 6" to 11" gravel underdrain system with a 6" perforated pipe collection system.

The sand bed depth recommended above should be considered the final consolidated depth. Depending on moisture content and compactive effort, a 5% increase in depth should be considered. The sand and gravel should be separated by a layer of permeable geotextile fabric, meeting the recommended specifications. Some authorities recommend not using geotextile fabrics to separate layers. Four inches of pea gravel may be substituted for the filter cloth. This allows for an integrated sand/gravel boundary with a higher matrix potential which allows easier water flow from the sand to the gravel. An alternative sand bed profile consists of a top layer of 12" to 18" of 0.02"-0.04" diameter sand. Lateral pipes are placed in trenches and covered with gravel and geotextile fabric. The laterals are underlain by a layer of drainage matting which provides for adequate vertical and horizontal flow.

FIGURE 5.13: SAND BED CROSS SECTIONS



The underdrain system consists of perforated collector and lateral pipe system. Perforations should be 3/8" diameter and should be spaced approximately 6" on-center. The lateral pipes should be spaced at a maximum distance of 10' on-center. Pipes should be adequate to accommodate the weight of the sand and gravel above. Pipes should be 6" PVC, Schedule 40 or greater. The entire underdrain system should have positive drainage, a design slope of at least .5% is recommended. A vertical standpipe should be provided for inspection and cleanout.

The outlet structure may simply be a direct connection of the underdrain piping system to a downstream storm drainage system, channel or waterway. In many cases the outlet structure will be a separate chamber, into which the underdrain system flows. This chamber will then discharge directly to the receiving waters. The outlet chamber may also act to collect overflow drainage associated with larger storm events, as applied in the underground sand filter and the perimeter sand filter.

5.7 SAND AND ORGANIC FILTER SYSTEM MATERIAL SPECIFICATIONS

Specifications are listed in Table 5.5 for many materials frequently used in sand and organic filters. These are typically specifications for materials required for filter practices, alternative localized specifications are available in different jurisdictions.

TABLE 5.5: MATERIAL SPECIFICATIONS

Parameter	Specification	Size	Notes
Sand	Clean AASHTO M-6/ASTM C-33 medium aggregate concrete sand	.02" - .04"	
Peat	Ash content: < 15% pH range: 5.2 - 4.9 Loose bulk density: .12 - .15g/cc		The material must be Reed-Sedge Hemic Peat, shredded, uncompacted, uniform, and clean.
Leaf Compost			Refer to CFS Treatment Systems, Inc (Stewart, 1992).
Underdrain Gravel	AASHTO M-43	½" - 2"	
Geotextile Fabric Between Layers	ASTM D-751 (Puncture Strength - 125 lbs.) ASTM D-1117 (Mullen Burst Strength-400psi) ASTM D-1682 (Tensile Strength - 300 lbs.)	0.8" Thick Equiv. Opening Size - # 80 U.S. Sieve	Maintain 125 gal/min per sq. ft. flow rate.
Impermeable Liner	ASTM D 751 (Thickness) ASTM D 412 (Tensile Strength - 1100 lbs, elongation - 200%) ASTM D 624 (Tear Resistance - 150 lbs/in) ASTM D 471 (Water Absorption - +8 to -2% mass)	30 mil thickness	Liner should be ultraviolet resistant. A geotextile fabric should be used to protect the liner from puncture.
PVC Piping	AASHTO M-278	6" - Rigid Schedule 40	⅜" perf. @ 6" centers, 4 holes per row .

5.8 CONSTRUCTION ELEMENTS

Several specific considerations are important for the construction of sand and organic filtering practices. These include the following:

- Sufficient access to the basin for construction and maintenance is necessary. An access ramp should be provided with a maximum slope of 10% for vegetated ramps, 15% if the slope is stabilized with crushed stone or, 25% if paved.

- Provisions must be made for the removal of sediment (both from the sedimentation basin and filter bed chambers) either on-site in a pre-established location or off-site at an approved and permitted location.

- No runoff should enter the sand filter bed until the upstream drainage area is completely stabilized and site construction is completed. The sedimentation basin may serve as a temporary sediment control basin during site construction with the provision that overflows will bypass the filtration bed. The erosion and sediment control plan must be carefully designed and sequenced to allow for the construction of the filter bed while maintaining erosion and sediment control.

- The top of the filter bed must be constructed **completely level**. Allowance for settlement after initial construction is also required. A geotechnical engineer should specify a minimum and maximum compactive effort based on material (sand, peat, or compost) gradation, moisture content, thickness of the filter bed and design permeability.

- Materials used for construction should meet the specifications outlined in Table 5.5. Materials which might be damaged during construction (such as perforated PVC piping, geotextile liners, etc.) should be stored in a safe location and handled carefully. Exposed piping and accessories should be constructed out of durable, strong materials to avoid susceptibility to damage by vandalism.

- Underground sand filters, facilities within sensitive groundwater aquifers, and filters designed to serve urban hotspots should be tested for water tightness prior to placement of filter layers. Entrances and exits should be plugged and the system completely filled with water to demonstrate water tightness.

- Overflow weirs, multiple orifices and flow distribution slots must be constructed completely level to ensure adequate distribution of design flows.

- Access manholes and/or grates to underground and below grade structures should be provided for each subsurface chamber. Manholes should be in compliance with the standard specifications of the relevant jurisdiction. Manhole

diameters should be 30" to meet confined space access criteria (and not be too heavy to manually remove). Aluminum and steel louvered doors provide excellent access, light and ventilation for routine maintenance operations. Manhole steps should be placed to allow maintenance personnel easy access to structure bottoms. A 5' minimum height clearance (from the top of the sand layer to the bottom of slab) is required for all fixed permanent underground structures. Lifting rings or other suitable element should be provided to lift and replace structure top slabs.

- The main collector pipe for underdrain systems should be constructed at a minimum slope of 0.5%. Observation and clean-out pipes must be provided for all underdrain piping.

- The underground sand filter should be constructed with a dewatering gate valve located just above the top of the sand filter bed. Should the filter bed and/or underdrain system clog completely, the gate valve can be opened to dewater the filter chamber for needed maintenance.

- To help extend the design life of the sand filter bed for the underground sand filter a wide mesh geotextile screen should be placed on the surface of the filter bed to trap the large quantities of trash, litter and organic detritus associated with highly urban areas. During maintenance operations the screen is rolled up, removed and cleaned, and reinstalled.

- Designers specifying a grass cover crop for sand or organic filter beds should choose an appropriate species which will develop a root system which does not inhibit infiltration. Appendix B describes several characteristics of grass. To help ensure that the filter bed will resist clogging on the pocket sand filter, a pea gravel "window" is recommended to cover approximately 10% of the sand bed surface area.

- Many of the alternatives call for the use of filter fabric to separate different layers of filter medium. These filter fabric layers are often the first place to clog with fine sediments. A 4" pea gravel layer may be substituted for filter cloth to separate layers of different materials.

- Whenever possible, sand filters should be visible so that they are easily recognizable as BMPs and can be quickly located for routine inspections. Perhaps the biggest concern with underground facilities is that they are often forgotten and inspections and maintenance are rarely performed.

5.9 MAINTENANCE

Several maintenance considerations are provided below. Table 5.6 presents a recommended inspector's checklist for stormwater sand filters.

5.9A GENERAL MAINTENANCE ELEMENTS

SEDIMENTATION BASIN

- The sedimentation basin should be cleaned out when the sediment depth exceeds 12". Removal of accumulated paper, trash and debris should be conducted every six months or after major storms.

- Vegetation growing within the sedimentation basin should be limited to 18" in height.

- Corrective maintenance is required for draw-down times exceeding 36 hours (24 hours is the design value). The perforated standpipe or low flow orifices should be checked and cleaned as necessary.

- Corrective maintenance is necessary for the sediment trap, when provided, if it does not drain within 96 hours.

- Access manholes, gate valves, flumes and other facilities shall be kept clean and ready for use.

FILTRATION BED COMPONENTS

- Grass clippings and other organic debris from landscape areas on the catchment should be bagged and removed from the site to prevent them from washing into and contaminating the sediment and filter chambers.

- Removal of silt should be conducted when accumulation exceeds approximately one-half (1/2) inch. When the filter layer will no longer draw down within the design period, the top layer of sand or organic media, sacrificial failure zone, or ballast gravel must be removed and replaced with new materials conforming to the original specifications. Any discolored or contaminated material, below the surface shall also be removed and replaced.

- Each sand or organic media filter should be inspected in accordance with the guidance in Table 5.6. Materials deposited on the surface of the filter chamber (e.g., trash and litter) should be removed manually . When the capacity of the filter bed begins to diminish due to surface clogging, manual removal of the top

few inches of discolored material should be done. In some cases, manual manipulation or roto-tilling of the surface may restore filtration capacity. Removed material should be replaced with fresh sand or organic media meeting the original design specifications. The contaminated material should be dewatered and disposed of at a pre-approved and permitted location.

- Urban hotspot land uses, particularly automotive uses with heavy oil/grease loadings, should conduct semi-annual clean-out of the sedimentation chamber and more frequent inspection of the filter bed.

- Vegetation growing within the basin should not exceed 18" in height.

5.9B SPECIFIC MAINTENANCE ELEMENTS

UNDERGROUND SAND FILTER

- The water level in the filter chamber should be monitored on a quarterly basis and after large storms for the first year of service. A log should be maintained documenting the results of the rate of dewatering and water depth of each observation. After the first year, monitoring may be reduced to a semiannual basis.

- The sedimentation chamber must be pumped out when the sediment depth reaches 12". Oil on the surface should be removed separately and recycled, the remaining material may be removed by vacuum pump and disposed of at an approved and permitted site.

PERIMETER SAND FILTER

- During the first year of operation, the system should be inspected after each major storm to ensure that the system is functioning properly. Inspections may be reduced to a semiannual basis afterwards.

- Trash collected on the grates protecting the inlets should be removed on a regular basis to preserve the inlet capacity of the facility.

PEAT-SAND FILTER SYSTEM

- Periodic mowing is required for the grass cover crop of the peat-sand filter bed. Grass clippings should be removed. Mowing frequency is largely up to the owner of the system, lower cutting height (less than 6") can be achieved by a conventional rotary lawn mower with a grass catcher. Mowing may need to be

as frequent as weekly during the peak growing season. Higher cutting levels can be achieved with a sickle mower, but grass raking will be required. Mowing using this method may only be required 3 to 4 times per year (Galli, 1990).

- Regular inspection should be conducted, particularly in the first year of operation, to ensure that the filter surface is not scouring or otherwise failing. Reseeding of areas sparsely covered with grass may be required.

COMPOST FILTER SYSTEM

- Annual maintenance of the compost filter bed consists of removing an accumulated sediment layer from the surface of the filtration bed and roto-tilling the compost media itself.

- The compost bed should be replaced with fresh compost every 3 to 4 years, or as heavy metal concentrations within the compost media exceed EPA's 503 Sewage Sludge Regulations for "clean sludge."

TABLE 5.6: RECOMMENDED INSPECTION CHECKLIST FOR STORMWATER SAND FILTERS (ADAPTED FROM SHAVER AND BELL, 1996)

Inspection Item	*Inspection*	*Disposition*
Debris Cleanout Inlets and outlets clear of debris? Filtration facility clear of debris?	Quarterly	Identify areas requiring cleanout and severity of buildup.
Vegetation Drainage area to facility stable? Area mowed, and clippings removed? Cover vegetation less than 18"?	Monthly during growing season, Quarterly during non-growing season.	Identify evidence of erosion, vegetation needing mowing, or unstabilized areas.
Filter Bed Chamber Evidence of filter bed surface clogging? Drainage area to facility clear of oil/grease sources? Sediment buildup on surface less than 1 inch?	Semi-annual	Identify clogged filter bed, source area contributions, and actions required.
Sedimentation Chamber Permanent pool wet? Evidence of leaking? Sediment buildup less than 12 inches?	Semi-annual	Identify leaking chamber and sediment level, specify actions required.
Structural Components Evidence of structure deterioration? Inlet grates, pipes, etc in good condition? Evidence of spalling or cracking of concrete?	Annual	Identify problems, specify actions required.
Outlets/Overflow Spillway Evidence of clogging of outlet pipe? Evidence of downstream erosion? Evidence of underdrain piping failure?	Annual	Identify problems, specify actions required.

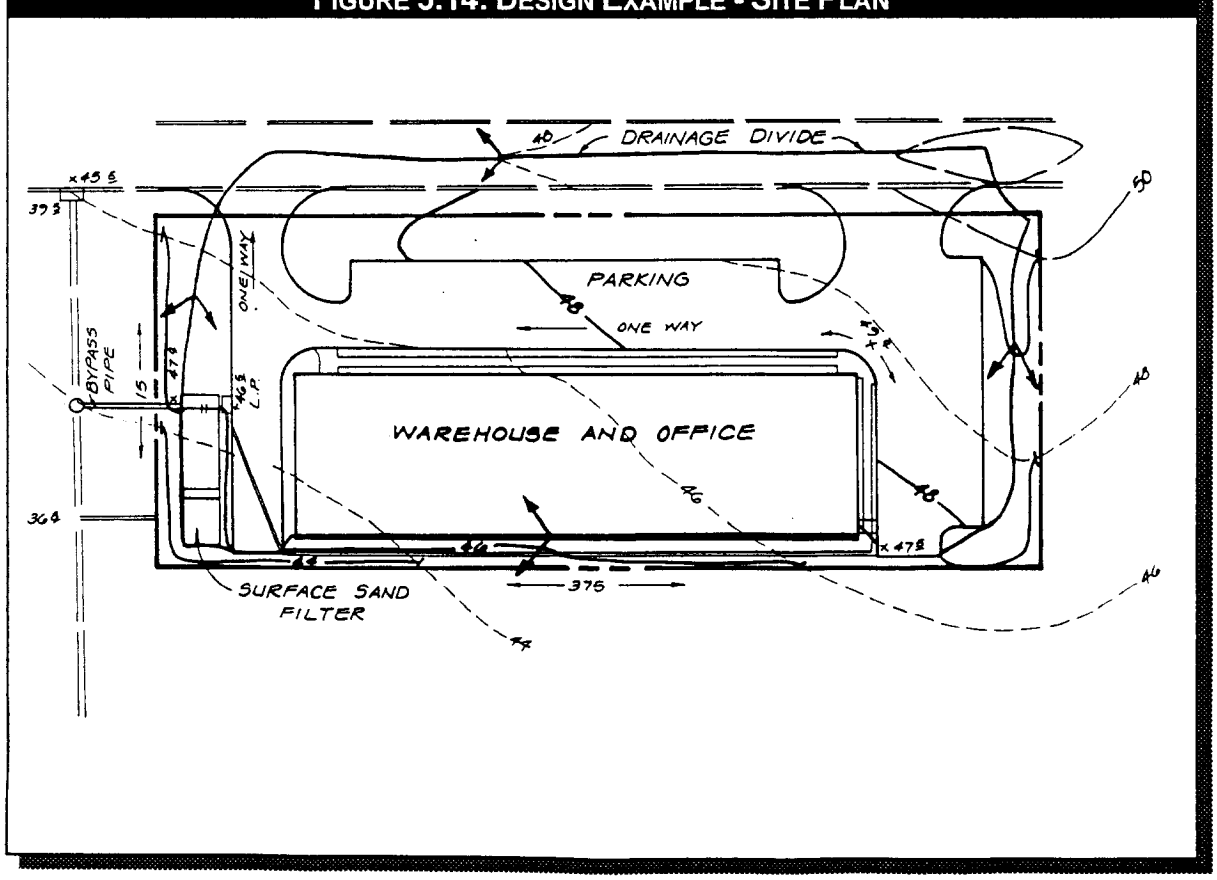
5.10 DESIGN EXAMPLE

Given: 1.35 acre site (see Figure 5.14)
Light Industrially zoned (predominately building & parking)
0.56 ac - paving
0.03 ac - sidewalk
0.39 ac - flat roof
0.02 ac - filter practice
0.35 ac - pervious

% Impervious = (0.56 + 0.03 + 0.39 + 0.02) / 1.35] * 100 = 74% *

* Note: impervious area < 75%, use equation 5.2 to size sedimentation chamber area (for I ≥ 75% use equation 5.2.1).

FIGURE 5.14: DESIGN EXAMPLE - SITE PLAN



1. Compute WQV

1.0" Rainfall (from Chapter 2, Section 2.7)
From Table 2.13

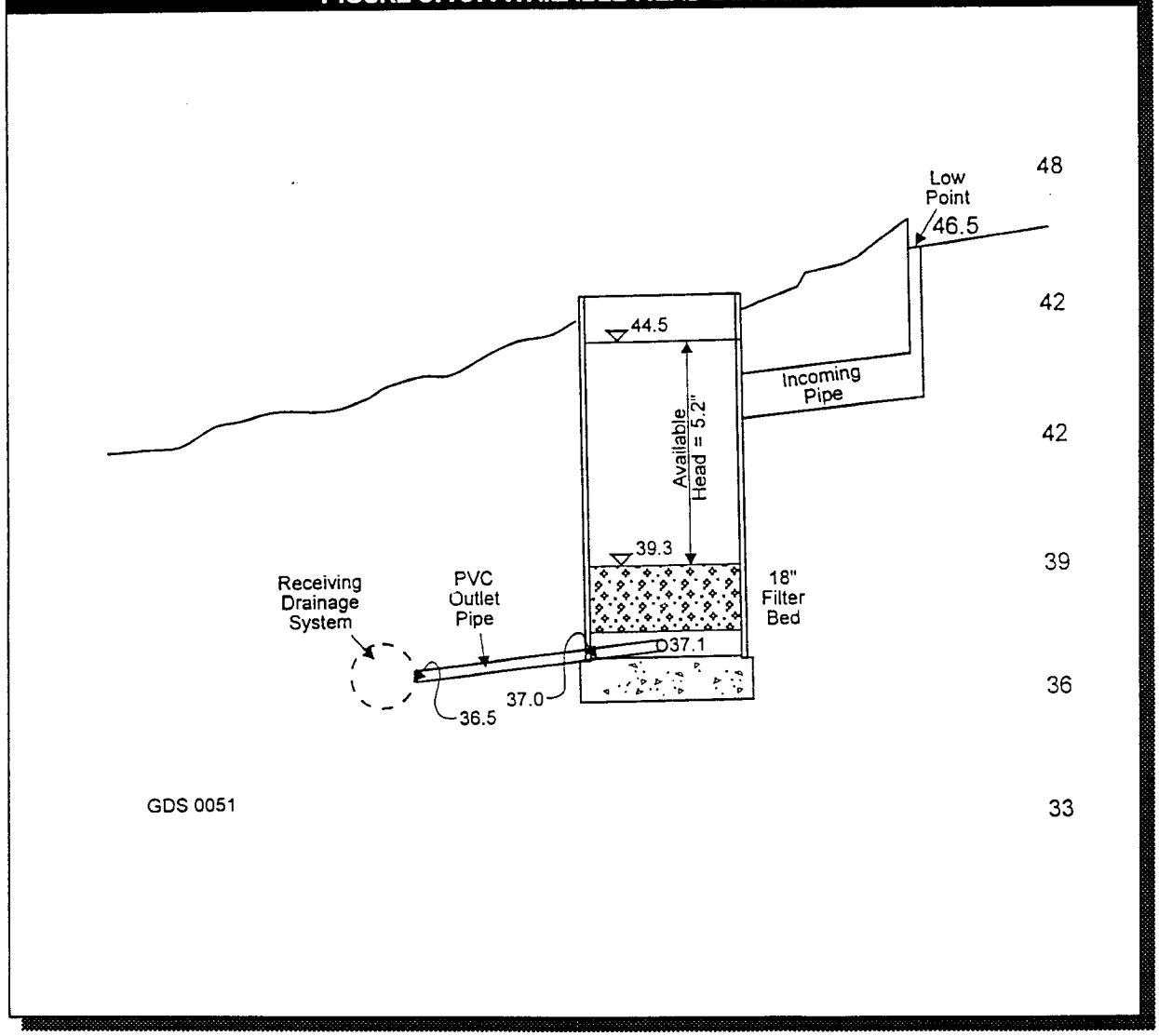
Flat roofs:	$R_v = 0.84$
Large impervious area:	$R_v = 0.97$
Small imp. area (streets):	$R_v = 0.70$
Filter surface area	$R_v = 1.00$
Pervious areas (silty soils)	$R_v = 0.11$

Weighted R_v = [0.56 (.97) + 0.03 (.70) + 0.39 (.84) + 0.02 (1.0) + 0.35 (.11)] / 1.35
$R_v = 0.70$

From Equation 2.2 WQV = 1.0" * 0.70 = 0.70"
WQV = 0.70" * (1.35 ac/ 12"/ft) * 43,560 ft^2/ac = 3,430 ft^3

Compute maximum head available (Figure 5.15)
Low point in street = el. 46.5 (subtract 2' to pass Q_{10} discharge) el. 44.5
Inv. @ storm drain system = el. 36.5
Inv. out of filter bed = el. 37.0
Top of filter bed = el. 39.3
allowable depth ($2 * h_f$) = 44.5 - 39.3 = 5.2 ft. Use <u>$2 * h_f = 5$ ft</u>

FIGURE 5.15: AVAILABLE HEAD DIAGRAM



2. Compute WQV peak discharge (Q_p)

From Chapter 2, Equation 2.3 (modified TR-55 methodology)

$CN = 1000 / [10 + 5P + 10Q - 10(Q^2 + 1.25\ QP)^{1/2}]$

$CN = 1000 / [10 + 5\ (1.0") + 10\ (.70") - 10\ ((.70")^2 + 1.25\ (.70")\ (1.0"))^{1/2}]$

$CN = 96.93$ Use <u>CN = 97</u>

Use $t_c = 0.1$ hr.

From TR-55, Chapter 4: I_a = 0.062, I_a/P = 0.092 / 1.0 = 0.09
From Exhibit 4 - II (TR-55) q_u = 1040 csm/in
For 1.0" rainfall
Q_p = 1040 csm/in (.70") (1.35 ac / 640 ac/mi²) = 1.5 cfs

Compute 10 year discharge for bypass
(conventional TR-55 methodology)

For 74% impervious, B soils (CN = 98 for Imp., CN = 61 for open space)
CN = .74 (98) + .26 (61) = 88.4 Use CN = 88
Use t_c = 0.1 hr.
From TR-55, Chapter 4: I_a = 0.273, P = 5.0", I_a/P = 0.273/5.0 = 0.055
From Exhibit 4-II, q_u = 1040 csm in
For 5.0" rainfall
Q_{10} = 1040 csm/in (3.67") (1.35 ac/640 ac/mi²) = 8.0 cfs

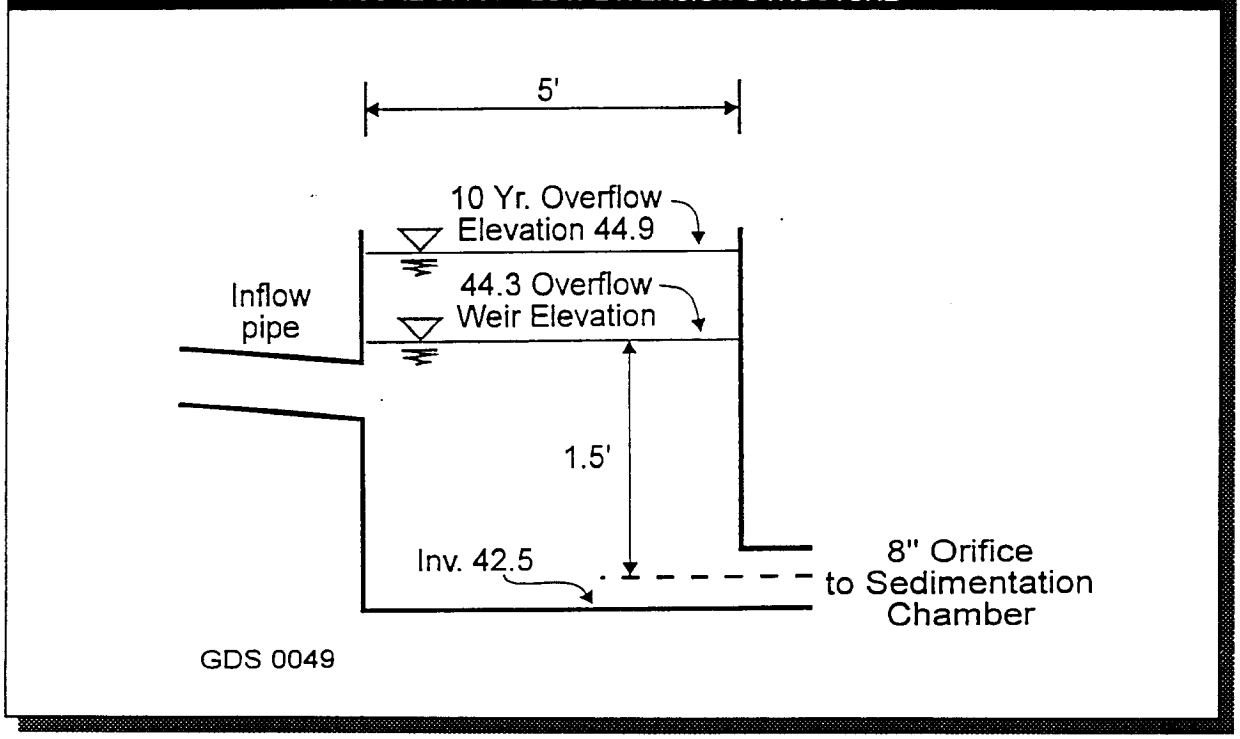
3. **Size flow diversion structure**
(see Figure 5.16)
Size low flow pipe to pass 1.5 cfs with 1.5' of head
$Q = C*A\ (2gh)^{1/2}$
1.5 cfs = 0.6 * A (2*32.2 ft/sec²*1.5')$^{1/2}$
A = 0.25 ft² = $\Pi d^2/4$: d = 0.57' Use 8" (over sized)

10 year overflow elevation = 44.3
Set low flow orifice inv. el. @ 44.3 -[1.5' + (½ *8" * 1 ft/12")] = 42.47
Set at el. 42.5

Compute overflow elevation in diversion structure (weir equation)
(10 year overflow = 8.0 cfs)
$Q = CLh^{3/2}$
8.0 cfs = 3.1 * 5.0 ft * $h^{3/2}$
h = 0.64 ft Elevation = 44.3 + 0.6 = 44.9

Size outlet pipe: with 2.0' of head.
$Q = C*A\ (2gh)^{1/2}$
8.0 cfs = 0.6 A (2*32.2 ft/sec²*2.0')$^{1/2}$
A = 1.17 ft² = $\Pi d^2/4$: d = 1.22' Use 15" RCP outlet
Set invert @ elev. 44.9 -[2.0' + (½ *15" * 1 ft/12") = 42.28: Use 42.3

FIGURE 5.16: FLOW DIVERSION STRUCTURE



4. Size sand filter bed

From Equation 5.4: $A_f = WQV * (d_f)/k * (h_f + d_f)(t_f)$

$A_f = 3{,}430\ ft^3 * (1.5') / [3.5 * (2.5 + 1.5) * (40\ hr/24\ hr/day)]$

$A_f = 220.5\ ft^2$ = 12' by 18.4': Use <u>12' by 20' (= 240 ft²)</u>

where: $d_f = 1.5'$

$h_f = 2.5'$

k = 3.5 ft/day (Table 5.4)

t = 40 hr

5. Size sedimentation chamber

From Equation 5.2 (Camp-Hazen equation) $A_s = 0.066 * (WQV)$

$A_s = 0.066 * (3{,}430\ ft^3) = 226\ ft^2$

for 12' width 226 ft² / 12' = 18.9 ft: Use <u>12' by 20 ft (= 240 ft²)</u>

6. **Compute V_{min} = ¾ * WQV**
   (Equation 5.3)

   V_{min} = ¾ * (3,430 ft³) = 2,573 ft³

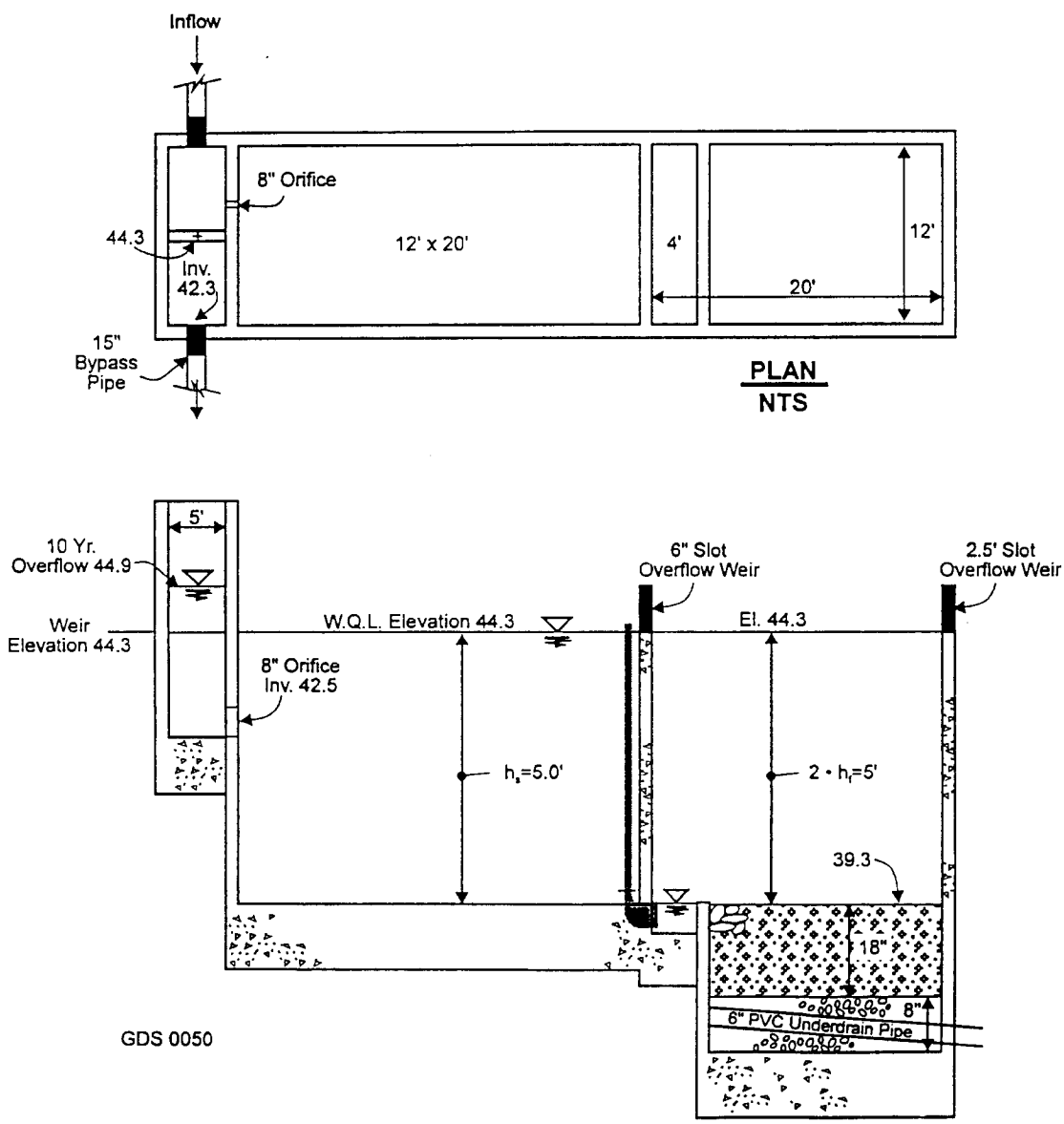
7. **Compute volume within practice**
   Compute volume within the filter bed (V_f): $V_f = A_f * (d_f) * (n)$
   V_f = 240 ft² * 2.0 * 0.4 = 192 ft³
   Compute temporary storage above filter bed ($V_{f\text{-temp}}$): $V_{f\text{-temp}} = 2 * h_f * A_f$
   $V_{f\text{-temp}}$ = 2 * 2.5' * 240 ft² = 1,200 ft³
   Compute remaining volume for sedimentation chamber (V_s):
   $V_s = V_{min} - (V_f + V_{f\text{-temp}})$
   V_s = 2,573 ft³ - (192 ft³ + 1,200 ft³) = 1,181 ft³ (note: V_s is approx 50% of V_{min})
   Compute height in sedimentation chamber (h_s): $h_s = V_s/A_s$
   h_s = 1,181 ft³ / 240 ft² = 4.9 ft set h_s = 5.0 ft (h_s>2 * h_f, and h_s > 3')
   (~~4.9~~' is less than available head of 5.2', OK)
   5.0'
   See Figure 5.17

8. **Compute overflow weir sizes:**

From sedimentation chamber (size to pass ⅔ of WQV peak discharge)
   Q_p = 2.0 cfs
   Weir equation: $Q_w = CLh^{3/2}$
   ⅔ * 2.0 cfs = 3.1 * L $(1.0)^{3/2}$, L = 0.4': Use <u>L = 0.5 ft</u>

From filter bed chamber (size to pass ⅓ of WQV peak discharge)
   ⅓ * 2.0 cfs = 3.1 * L $(0.2)^{3/2}$, L = 2.4': Use <u>L = 2.5 ft</u>

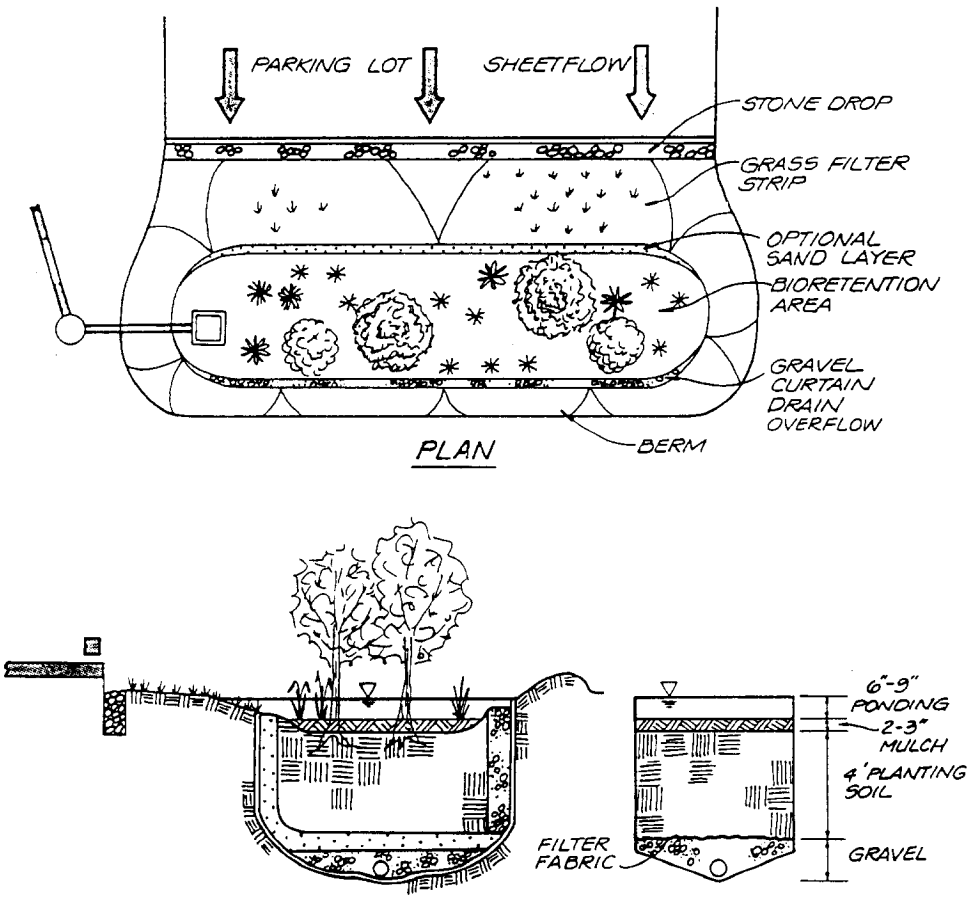
FIGURE 5.17: PLAN/PROFILE: SURFACE SAND FILTER



Chapter

6

Key Design Elements of Bioretention Systems



BIORETENTION FILTER

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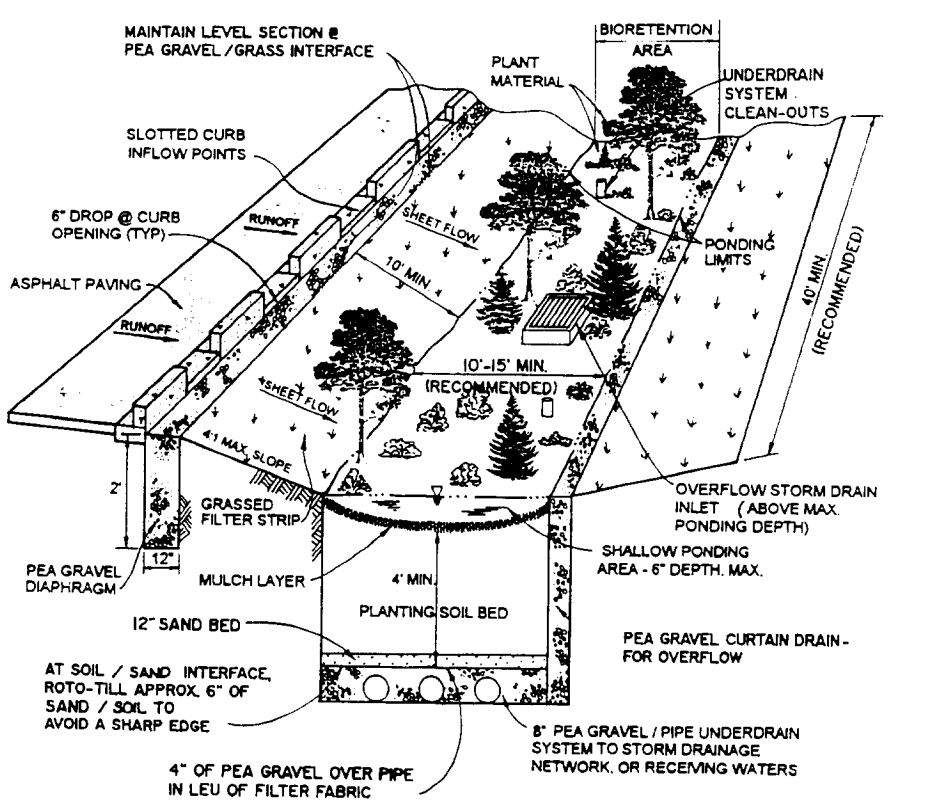
CHAPTER 6
KEY DESIGN ELEMENTS OF BIORETENTION SYSTEMS

6.1 INTRODUCTION

The bioretention concept was originally developed by the Prince George's County, Maryland, Department of Environmental Resources in the early 1990's as an alternative to traditional BMP structures (ETAB, 1993). Bioretention is a practice to manage and treat stormwater runoff using a conditioned planting soil bed and planting materials to filter runoff stored within a shallow depression. The method combines physical filtering and adsorption with biological processes. The system consists of a flow regulation structure, a pretreatment filter strip or grass channel, a sand bed, pea gravel overflow curtain drain, a shallow ponding area, a surface organic layer of mulch, a planting soil bed, plant material, a gravel underdrain system, and an overflow system (Figure 6.1).

FIGURE 6.1: THE BIORETENTION CONCEPT SYSTEM COMPONENTS



SOURCE: ADAPTED FROM PRINCE GEORGE'S COUNTY - DESIGN MANUAL FOR THE USE OF BIORETENTION IN STORMWATER MANAGEMENT, 1993

Bioretention is intended as a water quality control practice only and therefore should generally be located off-line. Several methods are presented for diversion of the Water Quality Volume (WQV) into these facilities. In some instances online applications may be appropriate where the drainage area is limited or where insufficient room is available to accomplish a diversion of the WQV. In these instances, the designer must accommodate the larger storms with sufficient erosion protection measures and adequate overflow provisions.

6.2 ALTERNATIVE APPLICATIONS

Bioretention can be applied to almost all development situations except perhaps the ultra-urban condition (where pervious surfaces are likely to be limited to 5% or less). The concept is applicable for residential land uses, either on private lots or within common open space, and is certainly applicable for treating parking lot runoff. The practice is also applicable for roadways where adequate space is available for off-line implementation. Finally, bioretention facilities are good candidates for pervious surface treatment, such as golf courses (See Chapter 3 and Figure 6.2).

6.3 SYSTEM COMPONENTS

Each component of the bioretention system is integral to the long term success of the practice and must be evaluated carefully in the overall design.

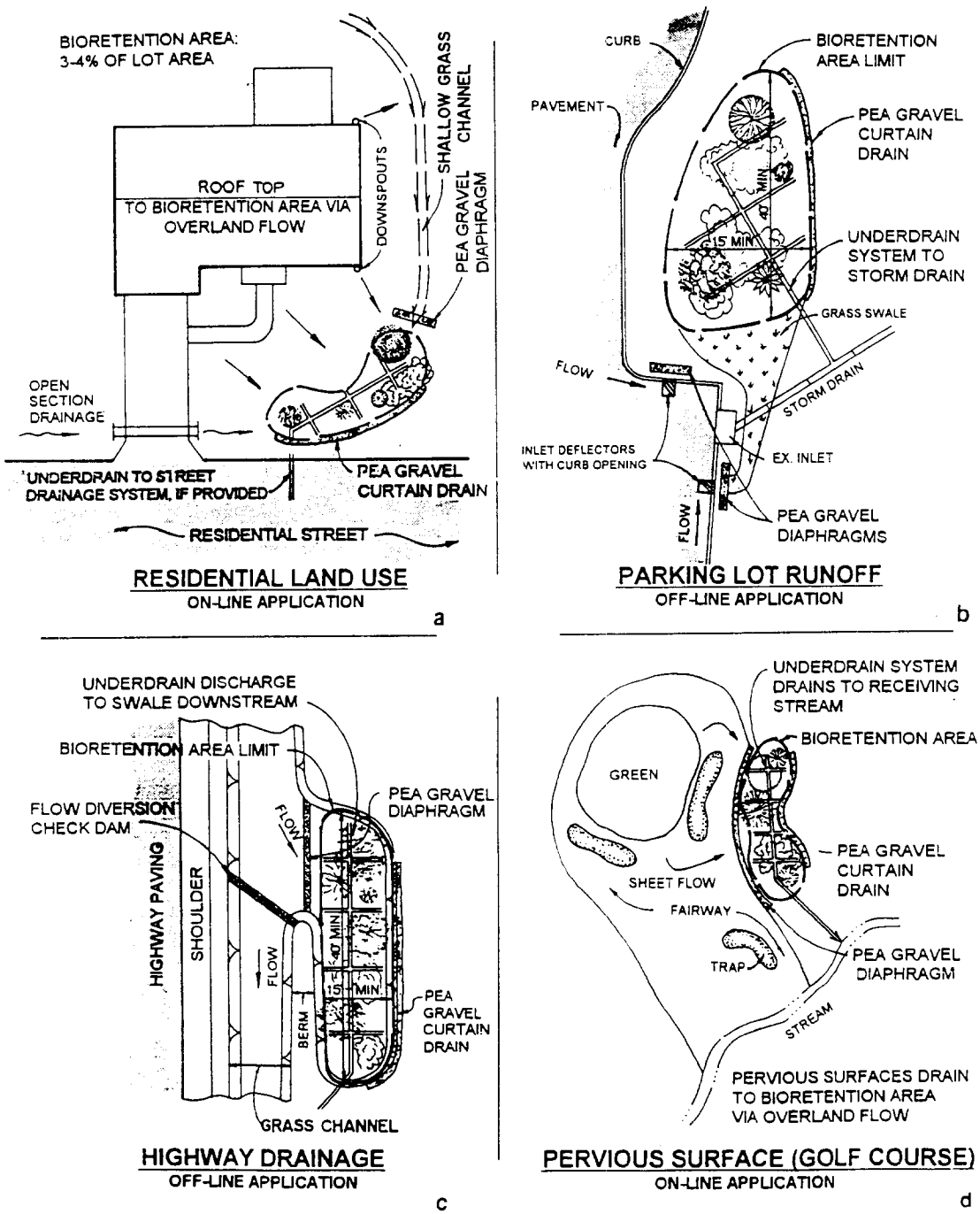
6.3A FLOW REGULATION AND/OR INTAKE STRUCTURE

For off-line applications, this element is responsible for ensuring that the WQV is captured and diverted to the practice for treatment. The isolation/diversion technique within the drainage system (described in Chapter 5) is one method for diverting the WQV to the bioretention system. Other principal techniques are described later in this chapter. The intake structure is equally important for both off-line and on-line applications to insure non-erosive velocities with adequate protection against clogging.

6.3B PRETREATMENT FILTER STRIP

This component is necessary to aid in reducing incoming velocities as well as capturing coarser sediment particles to extend the design life and reduce replacement maintenance of the bioretention system. The pretreatment method may incorporate other techniques, such as a sand or gravel diaphragm to aid in extending the design life of the practice.

FIGURE 6.1: THE BIORETENTION CONCEPT SYSTEM COMPONENTS



6.3C PEA GRAVEL OVERFLOW CURTAIN DRAIN

This element provides an overflow feature to help augment infiltration into the planting soil bed. This allows a greater portion of the WQV to be treated by the facility.

6.3D SHALLOW PONDING AREA

The shallow ponding area just above the mulch layer and vegetation root zone provides surface storage for a percentage of the WQV. This area also allows for particulate settling during the detention period allowing finer particles to settle on the surface of the mulch layer.

6.3E SURFACE MULCH LAYER

The mulch layer provides an environment for plant growth by maintaining moisture and allowing for the decomposition of organic matter. The surface layer acts as a filter for finer particles still in suspension and maintains an environment for the microbial community to help breakdown urban runoff pollutants.

6.3F PLANTING SOIL BED

The planting soil bed provides the region for water and nutrients for the planting material above. The voids within the soil provide additional storage for the WQV. The soil particles can adsorb various pollutants through cation exchange.

6.3G PLANTING MATERIAL

The plant material takes up some nutrients and other pollutants, and available water through evapotranspiration. The use of native plant material, combined with a minimum planting area size provides cover for wildlife and creates a micro-environment within the urban landscape.

6.3H SAND BED

The sand bed is provided to keep finer soil particles from washing out through the underdrain system, and it provides an aerobic sand filter as a final "polishing" treatment media.

6.3I GRAVEL UNDERDRAIN SYSTEM

This component is utilized to collect and distribute treated excess runoff. A properly designed underdrain system helps keep the soil from becoming saturated. The underdrain system consists of a gravel layer with a 4" or 6" perforated piping system (maintaining a 2" cover of gravel over the pipe).

6.3J OVERFLOW SYSTEM

The overflow system provides a means to convey larger storm flow volumes to the downstream receiving waters or drainage system. This component usually consists of a conventional drainage catchbasin, inlet, or overflow channel located slightly above the shallow ponding limit.

6.4 *FLOW REGULATION*

An off-line design is recommended for most bioretention applications. Larger storms are likely to cause erosion problems at the inflow points, disrupt the mulch layer, and otherwise negatively affect the plant material. For situations where it is not possible to separate the WQV from the larger storms an on-line design may be utilized. For these applications, it is imperative that adequate precautions are taken to protect the inlet, mulch layer, and plant material (e.g., stone stabilization or synthetic erosion protection materials). On-line designs should only be considered for very small drainage areas.

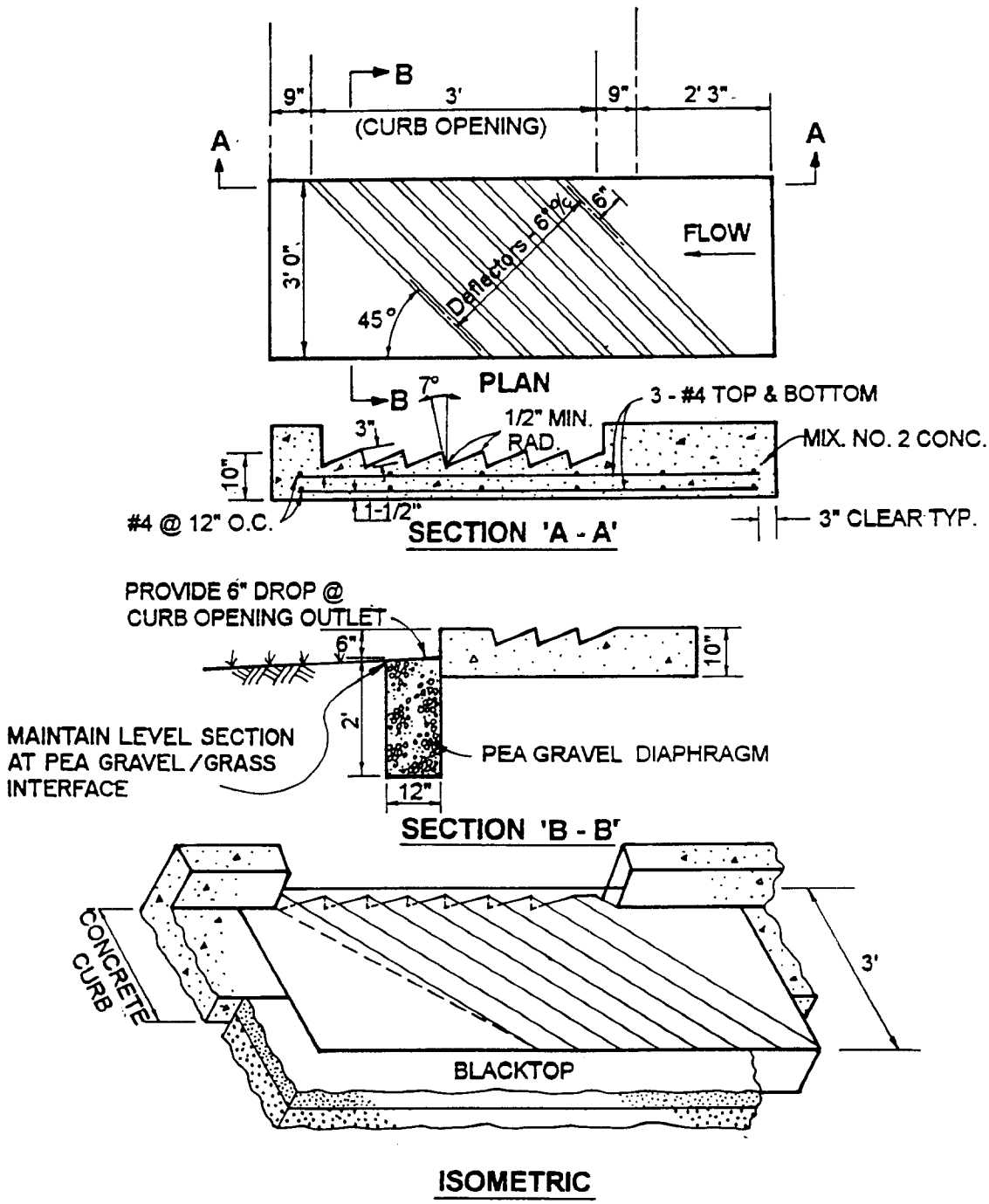
The basic flow regulation design objective is to capture and divert the WQV to the bioretention area and "bypass" the larger storms to the downstream storm drainage system, detention pond or receiving water. In some cases, utilizing bioretention structures for treating the WQV throughout a site or subcatchment may also provide significant runoff attenuation to effectively manage smaller "quantity control" storms as well, and therefore, the need for downstream detention facilities may be eliminated. Refer to the Chapter 2 discussion of the Rainfall Frequency Spectrum and stormwater management control points for more information. Table 6.1 presents several alternative techniques for diversion of the WQV.

TABLE 6.1: FOUR METHODS FOR WQV DIVERSION TO BIORETENTION FACILITIES

Divert runoff from an enclosed storm drain system using the peak flow rate (Q_p) methodology described in Chapter 5 (Figure 5.7).
Divert runoff from curbed pavements using a curb opening with slotted deflector grooves in the gutter pan (ETAB, 1993). Utilize a 6" drop below the curb, with a pea gravel diaphragm, as illustrated in Figures 6.2b and 6.3.
Divert runoff from curbed parking lots utilizing a slotted curb with limited width and design the parking area grades to divert the WQV into the bioretention area. Once the capacity of the slotted curb is exceeded, additional runoff bypasses the facility to flow into a downstream storm drain inlet or channel. This method utilizes a portion of a parking area for temporary ponding and may not be acceptable for areas with limited parking (Figure 6.4).
Divert runoff from an open conveyance channel into the bioretention area. A log, concrete curb stop or other structural measure in the form of a check dam backs-up flowing water to a 6" maximum depth which then flows into the adjacent bioretention area. Once the ponded water reaches the design capacity, the water overflows the checkdam and proceeds downstream. (See Figure 6.2c for application of this technique).

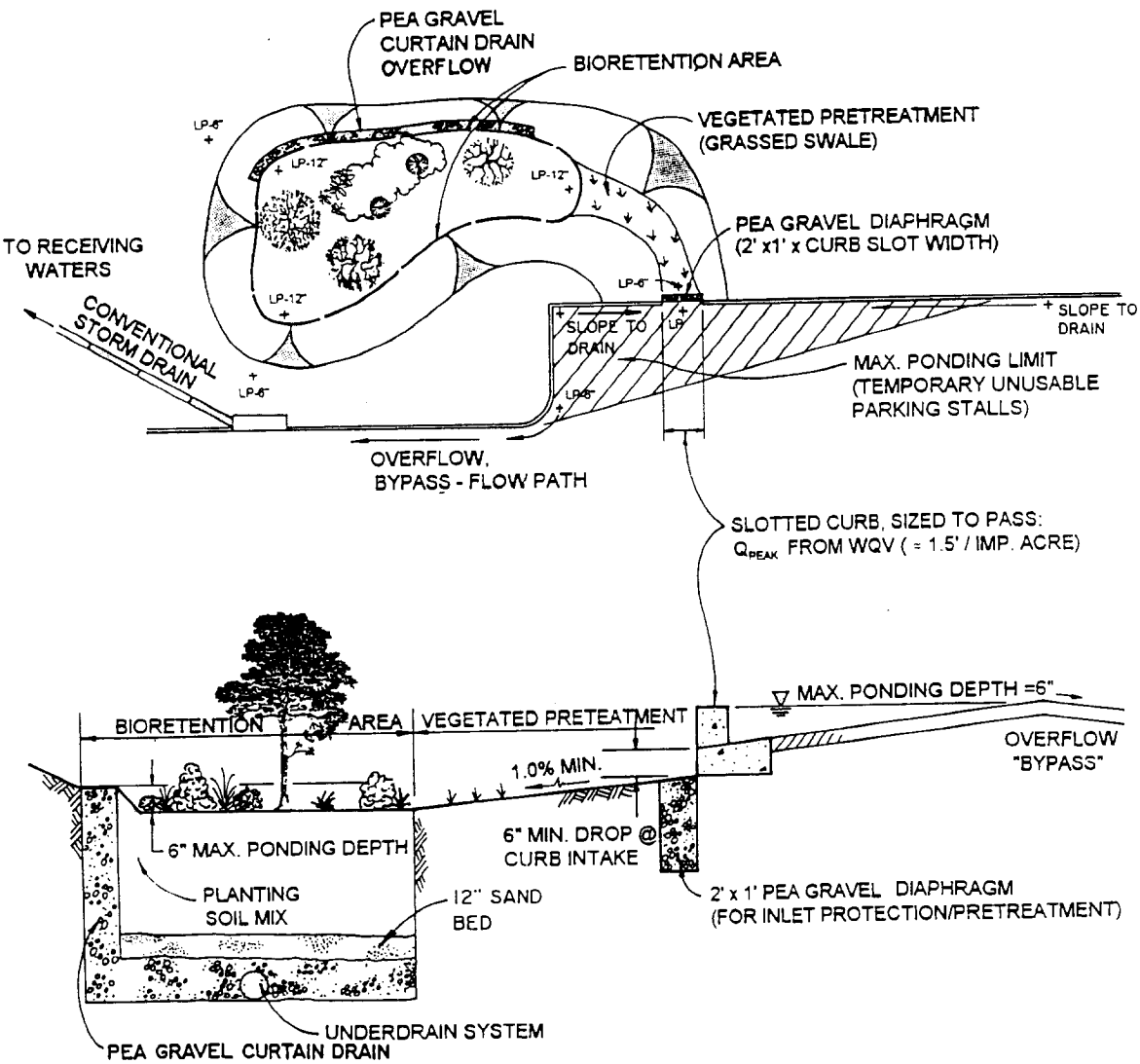
On-line design applications should be limited to a maximum drainage area of 0.5 acres (See Figure 6.2a for an example of this application). The designs should incorporate adequate overflow measures to accommodate larger flows. A yard inlet storm drainage structure, with the throat opening held 6" above the top of the mulch layer is one technique for handling overflow.

FIGURE 6.3: INLET DEFLECTOR DETAIL



SOURCE: ADAPTED FROM PRINCE GEORGE'S COUNTY DESIGN MANUAL FOR THE USE OF BIORETENTION IN STORMWATER MANAGEMENT, 1993

FIGURE 6.4: BIORETENTION AREA WITH SLOTTED CURB FLOW DIVERSION SYSTEM



SOURCE: ADAPTED FROM PRINCE GEORGE'S COUNTY - DESIGN MANUAL FOR THE USE OF BIORETENTION IN STORMWATER MANAGEMENT, 1993

6.5 PRETREATMENT

The primary pretreatment technique for bioretention facilities is through the use of a grass filter strip or grass channel. For applications where runoff enters the bioretention system through sheet flow, such as from parking lots, or residential back yards, a grass filter strip with a pea gravel diaphragm is the preferred pretreatment method. The length of the filter strip depends on the drainage area, imperviousness, and the filter strip slope. Table 6.2 gives some sizing guidelines as a function of inflow approach length, land use, and slope. The minimum filter strip length should be 10 feet.

TABLE 6.2: PRETREATMENT FILTER STRIP SIZING GUIDANCE

Parameter	*Impervious Parking Lots*				*Residential Lawns*				*Notes*
Maximum inflow approach length (feet)	35		75		75		150		
Filter strip slope	≤2%	≥2%	≤2%	≥2%	≤2%	≥2%	≤2%	≥2%	Maximum slope=6%
Filter strip minimum length	10'	15'	20'	25'	10'	12'	15'	18'	

For applications where concentrated (or channelized) runoff enters the bioretention system, such as through a slotted curb opening, a grassed channel with a pea gravel diaphragm is the preferred pretreatment method. The length of the grass channel depends on the drainage area, land use, and channel slope. Table 6.3 gives sizing for grass channels leading into a bioretention facility for a one acre drainage area. These values are based on approximately 10% of the stand alone BMP design criteria (see Chapter 7, Section 7.5: Grass Channel Design Procedure). The minimum grassed channel length should be 20 feet.

TABLE 6.3
PRETREATMENT GRASS CHANNEL SIZING GUIDANCE FOR A 1.0 ACRE DRAINAGE AREA

Parameter	*≤ 33% Impervious*		*Between 34% & 66% Impervious*		*≥ 67% Imperious*		*Notes*
Slope	≤2%	≥2%	≤2%	≥2%	≤2%	≥2%	Maximum slope = 4%
Grassed channel min. length (feet)	25	40	30	45	35	50	Assumes a 2' wide bottom width

The pea gravel diaphragm is designed to slow the velocity and aid in spreading out the flow entering the practice. In addition, this component captures the coarser-grained sediments. It is anticipated that the pea gravel diaphragm will exhibit clogging within the first three to four years after installation and may require periodic flushing and/or replacement. The maintenance schedule of the pretreatment measures are discussed further in this chapter.

6.6 FILTER MEDIA

Bioretention facilities are sized based on the consistent sizing criteria reviewed in Chapter 5 as derived from Darcy's Law by the City of Austin TX. (City of Austin, TX, 1988).

6.6A FILTER BED SURFACE AREA

Since the bioretention concept incorporates a gravel underdrain system and a porous soil filter medium and sand bed, runoff entering the shallow ponding area will slowly percolate through the soil bed in a fashion similar to other filter practices.

Equation 5.4 is utilized to estimate the minimum surface area and then the volume capacity is checked against that required to treat the WQV:

From Chapter 5:

$A_f = WQV * (d_f)/k * (h + d_f)(t_f)$

where:

A_f = Surface area of the bioretention planting bed (ft^2)
WQV = Water quality treatment volume (ft^3)
d_f = Planting soil bed depth (ft)
k = Coefficient of permeability for planting soil bed (ft/day)
h = Average height of water above the bioretention bed (ft); $h_{avg} = ½ * h_{max}$
t_f= Time required for the Water Quality Treatment Volume (WQV) to filter through the planting soil bed

- WQV is computed using the procedure outlined in Chapter 2
- d_f is four(4) feet
- k = 0.5 ft/day: Median value of a silt loam (Hwang, 1981) h is equal to 3", assuming a maximum ponding depth of 6" above the planting soil bed
- A value of 72 hours is recommended for the filter drawdown time (t_f)

Derivation of bioretention facility sizing criteria:

For a one (1) acre site which is 100% Impervious (R_v = 0.95)
WQV = [1.0"* (0.95)/ (12"/ft)]*(43,560 ft^2/ac) = 3,449 ft^3
k = 0.5 ft/day
d_f = 5' (4' soil + 1' sand bed)
h = 3" = 0.25'
t_f = 3 days
A_f = 3,449 ft^3* 5' /[(0.5 ft/day)*(5.25 ft)*(3 days)] = 2,190 ft^2
% of site area = 2,190/43,560* 100 = 5.0

Therefore, use the following equation for sizing the bioretention surface area:

$$A_f = D.A. * 5.0\% * R_v \qquad \textbf{Equation 6.1}$$

where,

A_f is the required surface area of the bioretention facility, D.A. is the drainage area and R_v is the volumetric runoff coefficient, which is computed using the methods outlined in Chapter 2.

6.6B FILTER BED WATER BALANCE EVALUATION

A water balance calculation was conducted to check the surface area sizing criteria stated above. The purpose of the calculation is to see how much runoff the system

can accommodate through temporary ponding, infiltration and evaporation verses how much runoff will by-pass the system. The calculation was evaluated over a 72 hour duration. 72 hours is the generally established maximum ponding time within infiltration practices in the Mid-Atlantic region (Maryland DNR, 1984). The water balance simulation is based on a spreadsheet computer program originally developed by Engineering Technologies Associates, Inc for Prince George's County for the development of the original bioretention design manual (ETAB, 1993).

The water balance computations are run at one-hour intervals for the 72 hour duration. The simulation infiltrates runoff from the ponded area to the planting soil/sand bed, and then into the gravel/pipe underdrain system. Because an underdrain is provided, the infiltration limitations of the in-situ soils are not evaluated. Runoff by-passes the bioretention area once the surface ponding volume is exceeded.

Percent of WQV captured by bioretention facility:

For a one (1) acre, 100 % impervious site, a bioretention facility should have a surface area as follows:

$A_f = (.95) * (43560) * (0.05) = 2{,}069\ ft^2$

Using a bioretention area with dimensions: 25' x 83' x 5'
8" gravel/pipe underdrain
12" wide gravel overflow curtain drain
6" deep shallow ponding surface area
12" sand bed
.5 ft/day (.25"/hr) infiltration rate for the planting soil bed
32.0 ft/day (16"/hr) infiltration rate for underdrain system
6 hour rainfall event

Results:
Approximately 70% of the WQV is accommodated by the bioretention facility (through surface ponding and infiltration). This corresponds well with the minimum volume required for sand and organic filter practices ($V_{min} = ¾ *$ WQV). The ponded surface volume infiltrates within 30 hours, but there is residual moisture within the soil planting bed after 72 hours.

In order to maintain a suitable micro-environment and to help simulate conditions which exist within an existing forest community, bioretention facilities must have a minimum area coverage. The sizing criteria presented above ensures the necessary treatment area and volume to accommodate the WQV, but additional criteria (Table 6.4) are necessary to assure the survival and success of the planted material.

TABLE 6.4: RECOMMENDED MINIMUM SIZING GUIDANCE FOR BIORETENTION FACILITIES (ADAPTED FROM ETAB, 1993)

- Minimum width of 10 feet
- Minimum length of 15 feet
- For widths greater than 10 feet, maintain a length to width ratio of 2:1
- Maximum shallow ponding depth of 6 inches
- Minimum planting soil bed depth of 4 feet (with 12" sand bed)

The minimum width allows for random spacing of trees and shrubs, it also permits planting densities which help create a micro-environment where stresses from urban stormwater pollutants are minimized. The 2:1 length to width ratio maintains a longer flowpath for the settlement of particulates and maximizes the edge-to-interior ratio. The maximum ponding depth of 6" provides surface storage for stormwater runoff (approximately 40% of WQV) but is not so deep as to adversely affect plant health. The 6" depth also will dissipate within a reasonable time (less than 3 days) which maintains flexibility in species selection, and minimizes the likelihood that the bioretention area will become a breeding ground for mosquitoes. The four foot planting soil bed depth is sized to provide adequate storage for the WQV, suitable capacity for root system growth and adequate moisture in the soil during dryer periods (ETAB, 1993).

6.6c PLANTING SOIL BED CHARACTERISTICS

The characteristics of the soil for the bioretention facility are perhaps as important as the facility location, size, and treatment volume. The soil must be permeable enough to allow runoff to filter through the media, while having characteristics

suitable to promote and sustain a robust vegetative cover crop. In addition, much of the nutrient pollutant uptake (nitrogen and phosphorus) is accomplished through adsorption and microbial activity within the soil profile. Therefore, the soils must balance soil chemistry and physical properties to support biotic communities above and below ground.

The planting soil should be a sandy loam, loamy sand, loam (USDA), or a loam/sand mix (should contain a minimum 35 to 60% sand, by volume). The clay content for these soils should by less than 25% by volume (EQR, 1996; ETAB, 1993). Soils should fall within the SM, ML, SC classifications or the Unified Soil Classification System (USCS). A permeability of at least 1.0 feet per day (0.5"/hr) is required (a conservative value of 0.5 feet per day is used for design). The soil should be free of stones, stumps, roots, or other woody material over 1" in diameter. Brush or seeds from noxious weeds, such as Johnson Grass, Mugwort, Nutsedge, and Canadian Thistle should not be present in the soils. Placement of the planting soil should be in lifts of 12 to 18", loosely compacted (tamped lightly with a dozer or backhoe bucket). The specific characteristics are presented in Table 6.5.

TABLE 6.5: PLANTING SOIL CHARACTERISTICS (ADAPTED FROM EQR, 1996; ETAB, 1993)

Parameter	*Value*
pH range	5.2 to 7.00
Organic matter	1.5 to 4.0%
Magnesium	35 lbs. per acre, minimum
Phosphorus (P_2O_5)	75 lbs. per acre, minimum
Potassium (K_2O)	85 lbs. per acre, minimum
Soluble salts	$\leq$ 500 ppm
Clay	10 to 25%
Silt	30 to 55%
Sand	35 to 60%

6.6D MULCH LAYER

The mulch layer plays an important role in the performance of the bioretention system. The mulch layer helps maintain soil moisture and avoids surface sealing which reduces permeability. Mulch helps prevent erosion, and provides a micro-environment suitable for soil biota at the mulch/soil interface. It also serves as a pretreatment layer, trapping the finer sediments which remain suspended after the primary pretreatment.

The Mulch layer should be standard landscape style, single or double, shredded hardwood mulch or chips. The mulch layer should be well aged (stockpiled or stored for at least 12 months), uniform in color, and free of other materials, such as weed seeds, soil, roots, etc. The mulch should be applied to a maximum depth of three inches. Grass clippings should not be used as a mulch material.

6.6E PLANTING PLAN GUIDANCE

Plant material selection should be based on the goal of simulating a terrestrial forested community of native species. Bioretention simulates an ecosystem consisting of an upland-oriented community dominated by trees, but having a distinct community, or sub-canopy, of understory trees, shrubs and herbaceous materials. The intent is to establish a diverse, dense plant cover to treat stormwater runoff and withstand urban stresses from insect and disease infestations, drought, temperature, wind, and exposure.

The proper selection and installation of plant materials is key to a successful system. There are essentially three zones within a bioretention facility (Figure 6.5). The lowest elevation supports plant species adapted to standing and fluctuating water levels. The middle elevation supports a slightly drier group of plants, but still tolerates fluctuating water levels. The outer edge is the highest elevation and generally supports plants adapted to dryer conditions.

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Appropriate plant materials for bioretention facilities are included in Appendix C. This list was adapted from the work by Prince George's County, Department of Environmental Resources, and their consultants (ETAB, 1993).

The layout of plant material should be flexible, but should follow the general principals described in Table 6.6. The objective is to have a system which resembles a random and natural plant layout, while maintaining optimal conditions for plant establishment and growth.

FIGURE 6.5: PLANTING ZONES FOR BIORETENTION FACILITIES

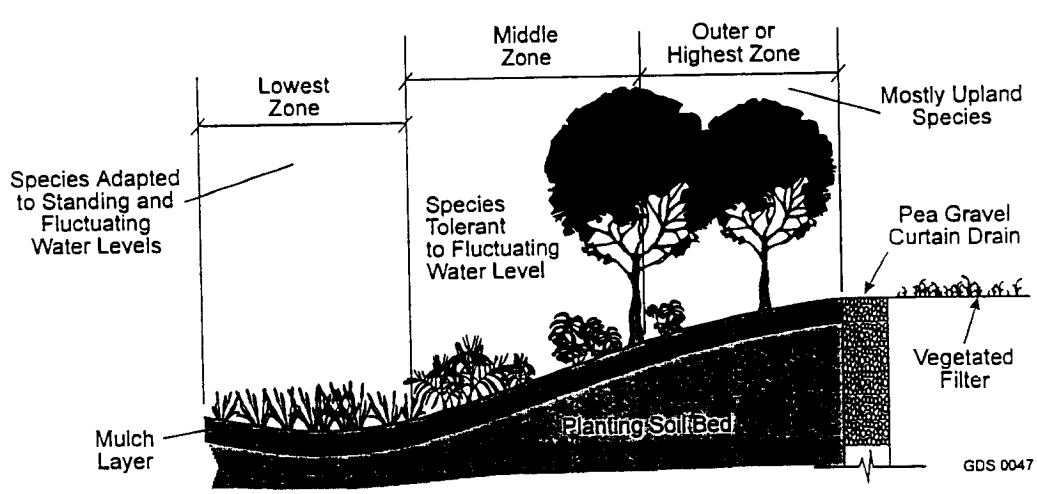


TABLE 6.6: PLANTING PLAN DESIGN CONSIDERATIONS

Native plant species should be specified over exotic or foreign species.
Appropriate vegetation should be selected based on the zone of hydric tolerance (see Figure 6.5).
Species layout should generally be random and natural.
A canopy should be established with an understory of shrubs and herbaceous materials.
Woody vegetation should not be specified in the vicinity of inflow locations.
Trees should be planted primarily along the perimeter of the bioretention area.
Urban stressors (e.g., wind, sun, exposure, insect and disease infestation, drought) should be considered when laying out the planting plan.
Noxious weeds should not be specified.
Aesthetics and visual characteristics should be a prime consideration.
Traffic and safety issues must be considered.
Existing and proposed utilities must be identified and considered.

6.6F PLANT MATERIAL GUIDANCE

Plant materials should conform to the American Standard Nursery Stock, published by the American Association of Nurserymen, and should be selected from certified, reputable nurseries. Planting specifications should be prepared by the designer and should include a sequence of construction, a description of the contractor's responsibilities, a planting schedule and installation specifications, initial maintenance, and a warranty period and expectations of plant survival. Table 6.7 presents some typical issues for planting specifications.

TABLE 6.7: PLANTING SPECIFICATION ISSUES FOR BIORETENTION AREAS

Specification Element	*Elements*
Sequence of Construction	Describe site preparation activities, soil amendments, etc.; address erosion and sediment control procedures; specify step-by-step procedure for plant installation through site clean-up.
Contractor's Responsibilities	Specify the contractors responsibilities, such as watering, care of plant material during transport, timeliness of installation, repairs due to vandalism, etc.
Planting Schedule and Specifications	Specify the materials to be installed, the type of materials (e.g., B&B, bare root, containerized); time of year of installations, sequence of installation of types of plants; fertilization, stabilization seeding, if required; watering and general care.
Maintenance	Specify inspection periods; mulching frequency (annual mulching is most common); removal and replacement of dead and diseased vegetation; treatment of diseased trees; watering schedule after initial installation (once per day for 14 days is common); repair and replacement of staking and wires.
Warranty	Specify the warranty period, the required survival rate, and expected condition of plant species at the end of the warranty period.

6.7 OVERFLOW

The overflow component of the bioretention system consists of the gravel underdrain system, pea gravel overflow curtain drain and a high flow overflow structure. The underdrain system should be designed in accordance with the principals reviewed in Chapter 5. These include: a 6" minimum perforated pipe system within an 8" gravel bed. The pipe should have ⅜" perforations, spaced at 6" centers, with a minimum of 4 holes per row. The pipe should be spaced at a maximum of 10' on-center and a minimum grade of 0.5% should be maintained. At least one cleanout per run should be provided. The underdrain system should be connected to the conventional drainage system, or should daylight to a suitable, non-erosive outfall.

The high flow overflow system usually consists of a yard drain catchbasin (see Figure 6.1), but any number of conventional drainage practices may be used, including an open vegetated or stabilized channel. The system should be designed to convey the peak discharge (Q_p) for the WQV, if the system is located off-line, and should be set above the shallow ponding limit. If the facility is located on-line, the high flow overflow should be designed as a conventional storm drainage structure, or channel. The overflow system should be connected to the site drainage system, or should outfall to a suitable, non-erosive location.

6.8 MATERIAL SPECIFICATIONS

Table 6.8 and 6.9 identify many of the material specifications necessary for bioretention facilities. Designers should refer to their local landscape specifications.

TABLE 6.8: MATERIAL SPECIFICATIONS

Parameter	*Specification*	*Size*	*Notes*
Planting Soil	Refer to Table 6.5	N/A	
Plantings	Refer to Table 6.9	Varies	Refer to Appendix C for specific information, by species.
Mulch	Shredded hardwood	N/A	Aged 2 to 12 months, minimum.
Pea gravel diaphragm and curtain drain	ASTM D 448 size no. 6	Varies (approximately⅛" - ¼")	Use clean bank-run river pea gravel.
Underdrain gravel	AASHTO M-43	½" - 2"	Use clean bank-run river pea gravel.
PVC piping	AASHTO M-278	6" - Rigid Schedule 40	⅜" perf. @ 6" centers, 4 holes per row.

TABLE 6.9: BIORETENTION PLANTING SPECIFICATIONS
(ADAPTED FROM EQR, 1996; ETAB, 1993)

Root stock of the plant material shall be kept moist during transport from the source to the job site.
Planting pits should follow LCA planting guidelines.
The diameter of the planting pit must be six inches larger than the diameter of the ball.
The planting pit shall be deep enough to allow ⅛th of the ball to be above existing ground. Tamp loose soil at the bottom of the pit by hand.
Set and maintain the plant straight during the entire planting process.
Backfill the pit with existing soil.
Trees shall be braced using 2" by 2" stakes only as necessary and for the first growing season only. Stakes are to be equally spaced on the outside of the tree ball.
Planting non-grass ground cover: ▸ Dig holes through the mulch with hand trowel, shovel, bulb planter, or hoe. ▸ Split biodegradable pots and remove non-biodegradable pots ▸ Surround the roots with soil below the mulch. Set potted plants so that the top of the pot is even with existing grade. Cover bare root plants to the crown. ▸ Thoroughly water the entire ground cover bed.
Grasses and legume seed shall be tilled into the soil to a depth of at least one inch. Grass and legume plugs shall be planted following the non-grass ground cover planting specifications.
No fertilization is necessary.

6.9 MAINTENANCE GUIDELINES

The following general maintenance guidance is recommended for bioretention systems. Although these systems are designed to simulate some of the functions of a natural forested plant community, the fact is, that these facilities are located

within an urban setting and will be exposed to a wide array of conditions, many of which will tend to compromise the effectiveness of the system. Bioretention facilities will require a reasonable amount of routine maintenance (not too different from conventional landscaping maintenance) to ensure that the system both functions well as a stormwater BMP, and maintains an aesthetic element compatible with the surrounding land uses.

Inspections are an integral part of any maintenance program. Bioretention facilities should be inspected on a semi-annual basis for the first year, and after major storm events. After the first year annual inspections should be sufficient. Since the practice is relatively new, longer term maintenance issues may become apparent which are currently not well understood. There are, however, several maintenance objectives common to all filtering practices, plus some common sense issues specific to bioretention facilities. The following is recommended:

6.9A PLANTING SOIL BED

- The soils of the planting bed should be tested on an annual basis for pH to establish acidic levels. If the pH is below 5.2, limestone should be applied. If the pH is above 7.0 to 8.0 iron sulfate plus sulfur can be added to reduce the pH.

- The soil bed may experience some erosion, particularly at the inflow points, periodic inspection and correction of erosion may be necessary.

- The surface of the bed may become clogged with fine sediments over time. Core aeration or cultivating of unvegetated areas may be required to ensure adequate filtration.

6.9B MULCH LAYER

- Bi-annual mulching, as part of a regular landscape contract, is recommended. The previous mulch may be removed and discarded to an appropriate disposal area or retained if it is decayed. The mulch should be placed to depths not to exceed 3". Seeded ground cover or grass areas should not receive mulching.

6.9C PLANTING MATERIALS

- Annual inspection of plant materials is necessary. Dead or severely diseased species should be replaced. Replacement of particular species should be

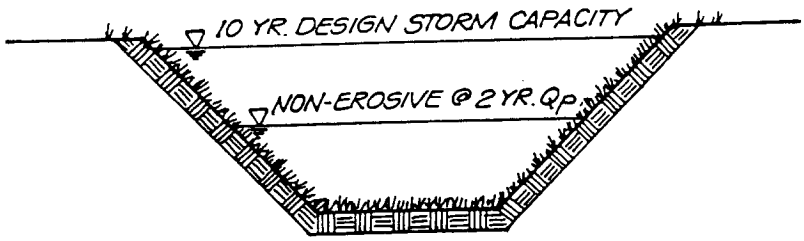
considered for species which fail to establish.

- Woody vegetation may require periodic pruning, depending on the adjacent land uses, to avoid conflicts with overhead utilities, or hazards with adjacent people and property. Pruning shall follow the standard pruning practices (ANSI A300, National Arborist Association, Inc., 1995).
- Remove plant stakes after the first growing season

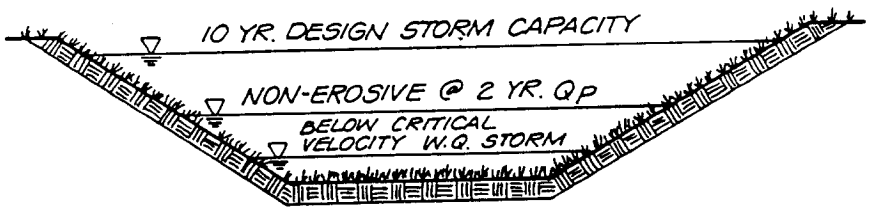
6.9D PRETREATMENT, INFLOW LOCATIONS, AND OVERFLOW

- The pea gravel diaphragm should be inspected annually for clogging. Sediment build-up should be removed, as needed. Replacement of the diaphragm after three to four years may be warranted (or when the voids are obviously filled with sediment and water is no longer infiltrating).
- The vegetated filter strip or grassed channel should be inspected for erosion rill or gulleys and corrected, as needed. Bare areas should be seeded, or sodded, as necessary.
- The inflow location should be inspected annually for clogging. Sediment build-up is common problem with many practices where runoff leaves an impervious surface and enters a vegetative or earthen surface. Any built-up sediment should be removed to avoid runoff by-passing the facility.
- The overflow structure should be inspected annually to ensure that it is functioning. Accumulated trash and debris should be removed, as necessary.

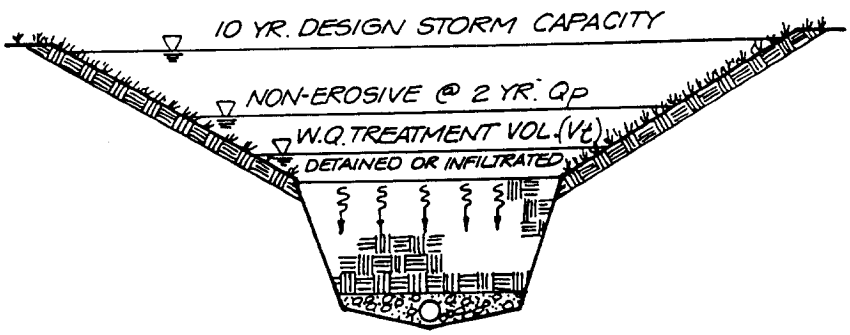
Key Design Elements of Open Vegetated Channels and Filter Strips



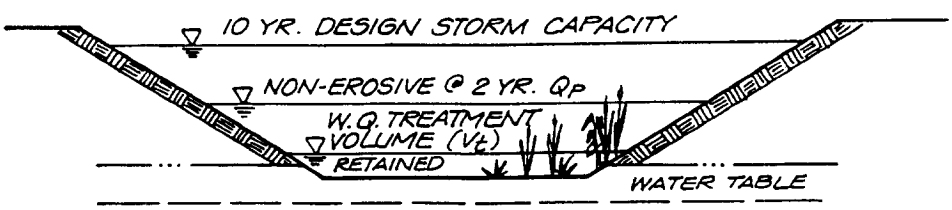
(a) DRAINAGE CHANNEL



(b) GRASS CHANNEL



(c) DRY SWALE



(d) WET SWALE

CHAPTER 7
KEY DESIGN ELEMENTS OF OPEN VEGETATED CHANNELS AND FILTER STRIPS

7.1 *INTRODUCTION*

This chapter explores the design principles of four different grass vegetative filter practices. Each of the four practices incorporate the four major design components discussed in Chapter 1: flow regulation, pretreatment, filtering, and overflow. Vegetative practices have been called a whole suite of names in the past. These include, grassy swales, bio swales, filter strips, grass buffers, and grass channels, to name a few. This chapter consolidates many of these past naming conventions and design principles into a unified approach for the design of water quality treatment using vegetative filters.

This chapter also reviews applicable material specifications and maintenance elements. Appendix B provides a detailed chart of various grasses and provides information to help assess the viability of each species for different design intents.

7.2 *DESIGN VARIATIONS*

Four basic design variations are presented here. The simplified design approaches and criteria, have been adapted from principles governing the design of open channels for conveyance purposes, and more recently, from principles governing the design of vegetative swales for water quality treatment (Horner, 1988). These practices are intended for application to smaller sites where the primary design objective is water quality treatment. The selection of the appropriate design variations are discussed in Chapter 3. The four basic design variations are briefly discussed below.

7.2A GRASS CHANNEL

The grass channel consists of a broad, mildly sloped open channel designed to maintain a minimum residence time of 10 minutes for the "water quality storm" (see Figure 7.1). Grass channels have traditionally been utilized only for stormwater conveyance purposes. In the past, designs ensured adequate capacity to carry a larger storm, usually the 10 year frequency storm and protection against erosion for smaller, more frequent storms, usually the 2 year event. Water quality treatment for the smallest, most frequent storms has only recently been a design consideration. The grass channel design is the only practice presented in this manual which uses a flow rate as the principle design criteria variable. This is referred to as a **rate based** design.

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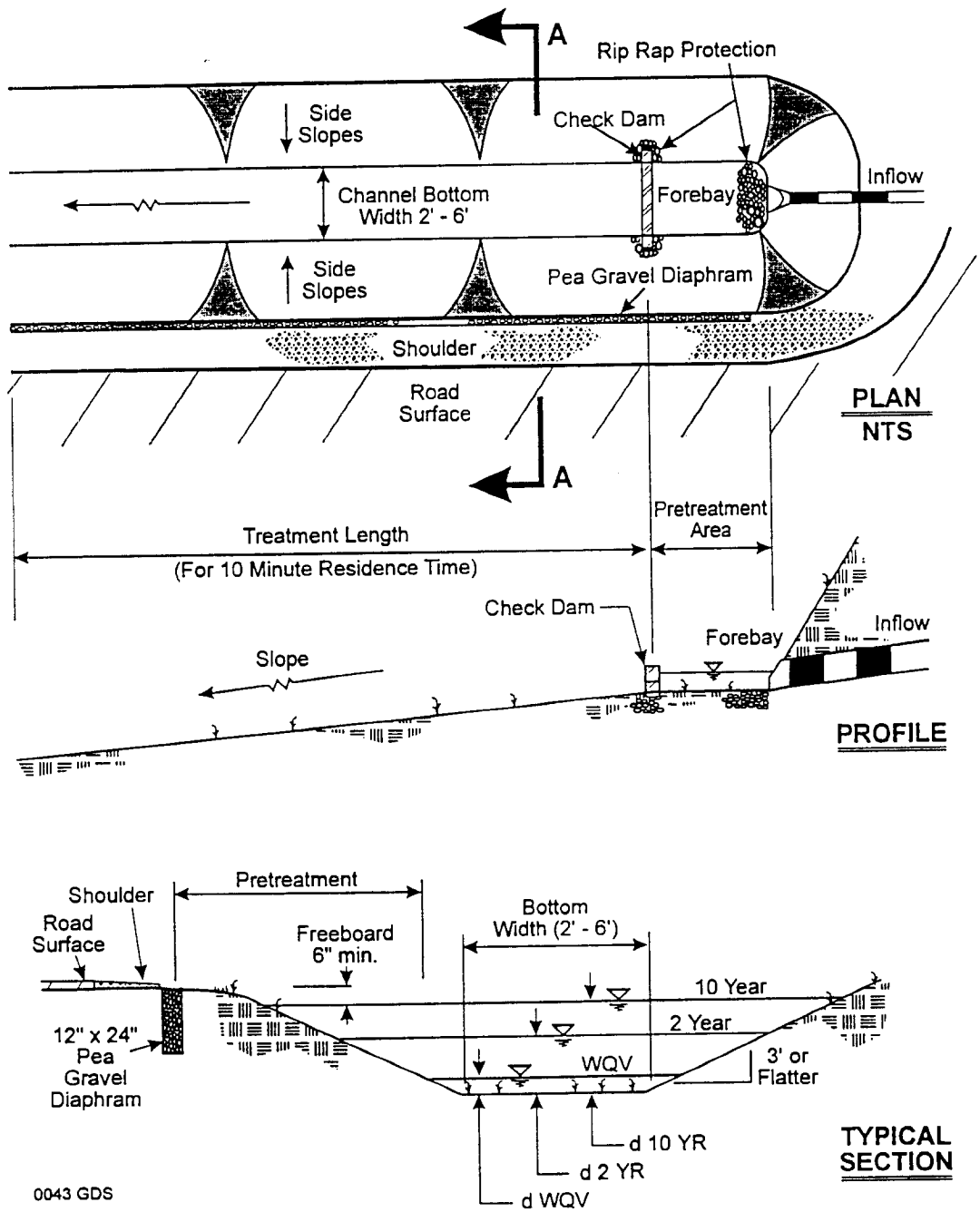
FIGURE 7.1: GRASS CHANNEL



7.2B DRY SWALE

The dry swale consists of an open channel capable of temporarily storing the water quality treatment volume (WQV) and a filtering medium consisting of a soil bed with an underdrain system. The dry swale uses a **volume based** sizing criteria. The dry swale is designed to drain down between storm events within approximately one day. The water quality treatment mechanisms are similar to bioretention practices except that the pollutant uptake is likely to be more limited since only a grass cover crop is available for nutrient uptake. Figure 7.2 illustrates the design components of the dry swale.

7.2C WET SWALE

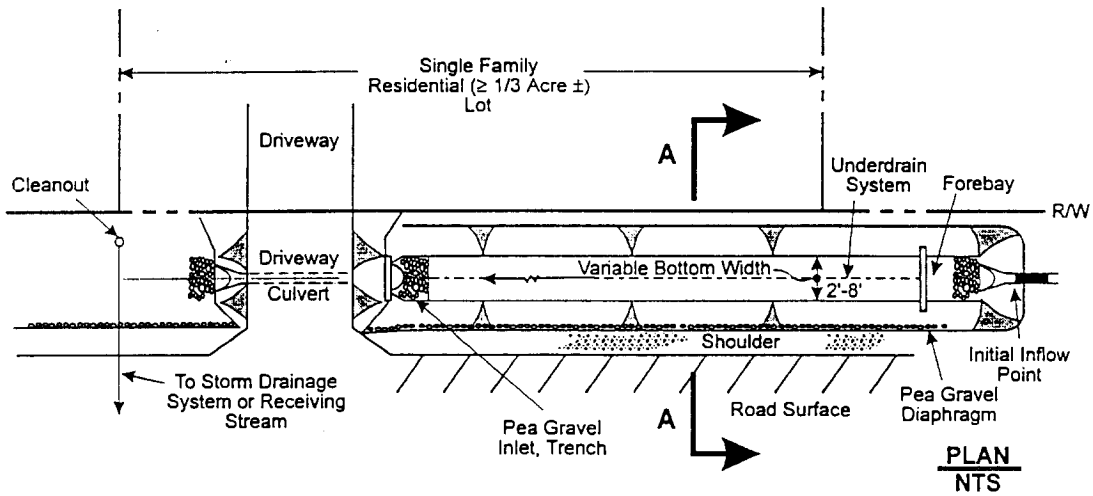
The wet swale also consists of a broad open channel capable of temporarily storing the WQV (also a **volume based** sizing criteria), but does not have an underlying filtering bed. The wet swale is constructed directly within existing soils and may or may not intercept the water table. Like the dry swale, the WQV within the wet swale should be stored for approximately 24 hours. The wet swale has water quality treatment mechanisms similar to stormwater wetlands which rely primarily on settling of suspended solids, adsorption, and uptake of pollutants by vegetative root systems. Figure 7.3 illustrates the design components of the wet swale.

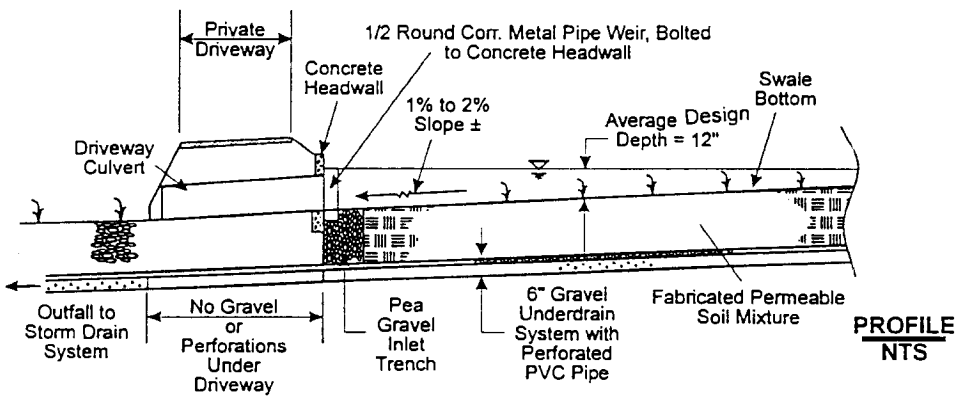
7.2D FILTER STRIP

Filter strips are grassed practices which accept sheet flow runoff from adjacent surfaces. Filter strips function by slowing runoff velocities and filtering out sediment and other pollutants. The design approach for filter strips involves site design techniques to maintain prescribed maximum sheet flow distances as well as checking to ensure adequate temporary storage for the WQV for a 24 hour period. Filter strips are also designed using a **volume based** sizing criteria.

As discussed in Chapter 1, it is doubtful that runoff can be maintained as sheet flow over distances beyond 150 feet for pervious surfaces, and 75 feet for paved surfaces. Once runoff concentrates, filtering is reduced or eliminated through short-circuiting, preventing effective treatment. Therefore, the use of filter strips to treat stormwater runoff is primarily a function of limiting the flow path to the filter. One of the main abuses of the past has been draining too much area through the filter strip. In most cases the sheet flow distance limitations will be the controlling factor. Figure 7.4 illustrates the primary design components of the filter strip.

FIGURE 7.2: DRY SWALE





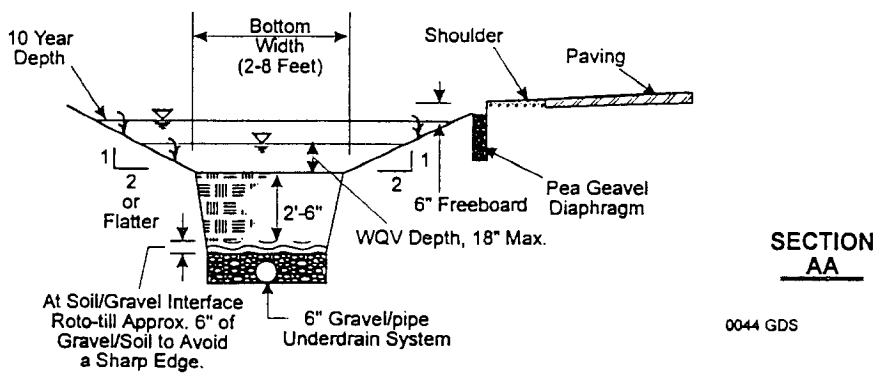
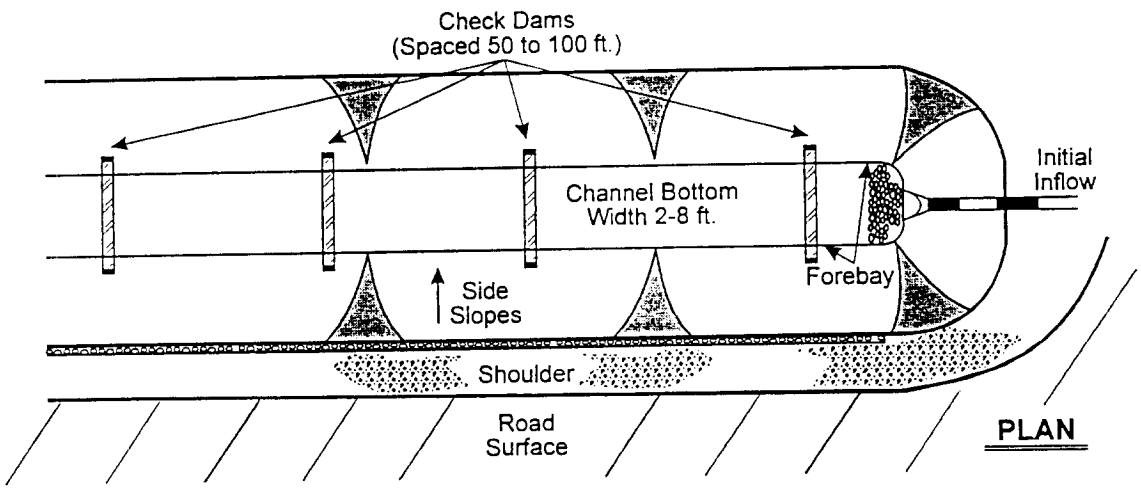
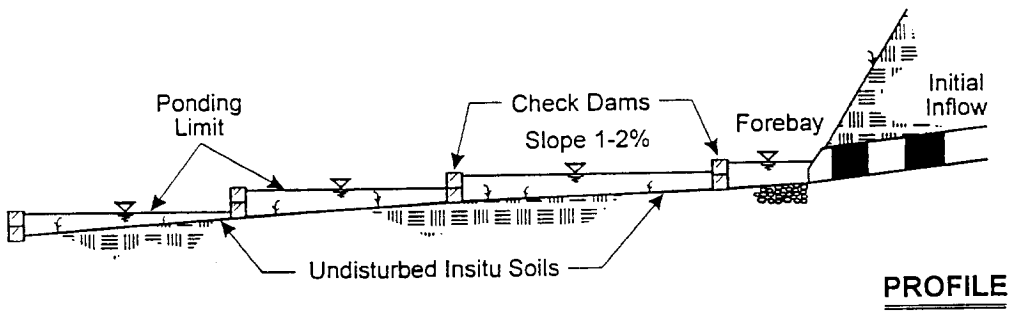


FIGURE 7.3: WET SWALE





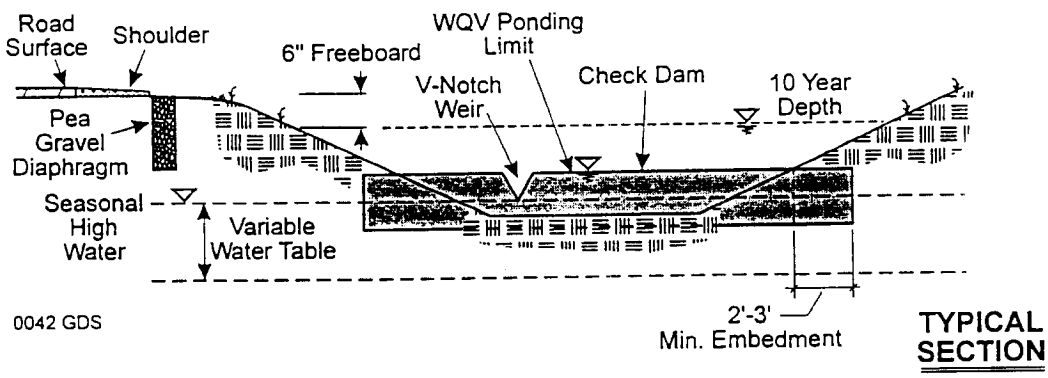
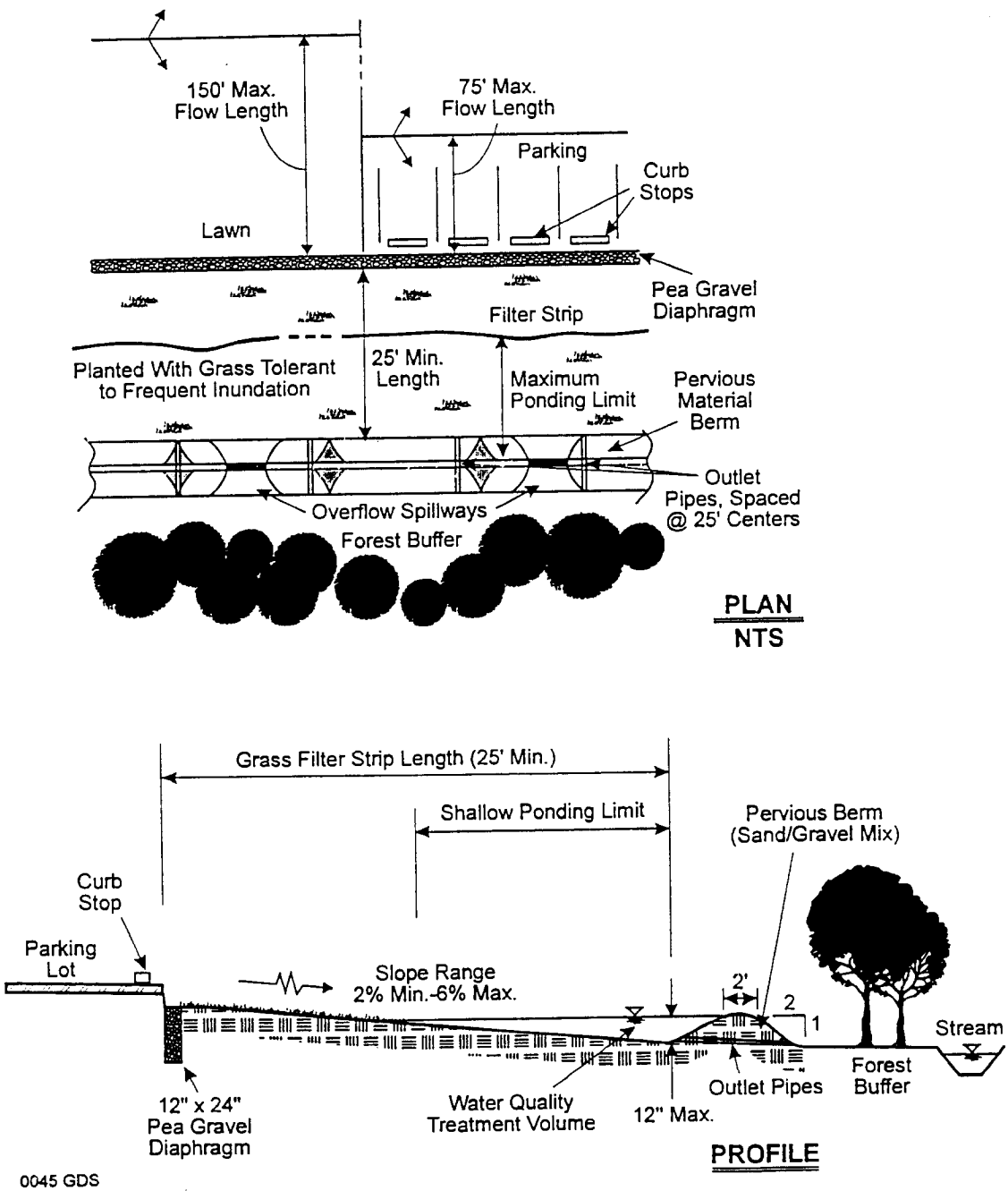


FIGURE 7.4: FILTER STRIP



7.3 FLOW REGULATION

The four design variations presented here are all primarily on-line stormwater treatment practices. The inherent nature of the practice and their applications for use do not lend themselves to many off-line applications. Clearly, it is still best to divert the WQV into the practice wherever possible, and bypass the larger storms around the facility. The grass channel, dry, and wet swales can receive runoff from concentrated sources (pipe outfalls), as well as from lateral sheet flow along the length of the practice. The isolation/diversion structure within the drainage network, reviewed in Chapter 5, is the preferred method for diverting concentrated flows, prior to entering these treatment practices.

The filter strip, which receives runoff through sheet flow from impervious or pervious surfaces is most commonly designed as an on-line practice. It may be possible, through site grading and other design techniques, to provide an overflow diversion which bypasses larger flows around the facility. However, since the filter strip drainage area is limited by the flow path, the volume of high flow runoff will not generally be excessive, and there should be little need to design the system as an off-line practice.

7.4 PRETREATMENT

As with all other filtering practices, pretreatment is necessary to extend the practice's functional life, as well as to increase the pollutant removal capability. All four design variations have incorporated nominal pretreatment as a component of the system design. The difference with these practices from other filtering practices is that the pretreatment component is more qualitative in nature and is an integral part of the practice itself (e.g., the side slopes of the grass channel). The design components for pretreatment which are specific to the four design variations are presented in Table 7.1. With the exception of sizing a forebay at the initial inflow point, there are no specific, quantitative sizing criteria for these pretreatment components.

TABLE 7.1: PRETREATMENT COMPONENTS FOR VEGETATIVE FILTERING PRACTICES

Grass Channel, Dry Swale and Wet Swale:

- A shallow forebay is provided at the initial inflow point of the channel. The volume of this forebay should equal approximately .05" per impervious acre of drainage.
- A pea gravel diaphragm is recommended along the top of the channel to provide pretreatment for lateral flows entering the practice.
- Mild side slopes (≤ 3:1) provide additional pretreatment for lateral flows.

Filter Strip:

- A pea gravel diaphragm is recommended along the top of the slope.
- The uphill area, above the shallow ponding limit provides additional pretreatment.

7.5 FILTER MEDIA

7.5A CHARACTERISTICS

The four vegetative filtering practices described in this chapter differ from the sand, organic, and bioretention practices because filtering is primarily through lateral or linear processes, as opposed to vertical filtering (with the exception of the dry swale which has a vertical component). For this reason, the sizing criteria is based on open channel design principles, for the grass channel; and volume detention principles for the remaining practices. Darcy's Law is not particularly applicable to this linear filtering process.

7.5B SIZING GUIDANCE

The grass channel, as previously stated, is a flow rate design based on open channel flow hydraulic characteristics. The principles of small storm hydrology, presented in Chapter 2, and the water quality pollutant removal processes, presented in Chapter 4, are used to design the channel. The dry swale, wet swale, and filter strip are all designed based on detention of the WQV.

GRASS CHANNEL DESIGN CONSIDERATIONS

The following design approach presents a three part criteria for sizing grass channels for stormwater quality treatment, while also accommodating larger storms. The channel is initially designed based on the treatment principles of Small Storm Hydrology for the Water Quality Storm (see Chapter 2), and then checked against the larger 2 year storm to ensure a non-erosive condition. Finally, the capacity for conveyance of the 10 year frequency storm is checked and a minimum freeboard is applied. The design procedure is a **rate based** sizing criteria which uses Manning's equation to compute velocities and depths based on specified channel geometry and slope.

The design application is predominately for highway drainage, but may also be appropriate for some residential applications and for treating small impervious areas (refer to Chapter 3). Figure 7.1 illustrates the design components of the grass channel.

The specific design considerations are presented below and summarized in Table 7.2.

Shape: The channel should be trapezoidal or parabolic in shape. The trapezoidal cross section is the easiest to construct and a more efficient hydraulic configuration. However, since channels tend to become parabolic in shape over time, a channel originally designed as a trapezoidal section should also be checked against parabolic sizing equations as a long term functional assessment. The criteria presented in this chapter assumes a trapezoidal cross section. Note that the same design principles will govern parabolic cross sections except for the cross sectional geometry.

Bottom width: For a trapezoidal cross section, size the bottom width between two and six feet. The two feet minimum allows for construction considerations and ensures a minimum filtering surface for water quality treatment. The six feet maximum prevents shallow flows from concentrating and potentially gullying, thereby maximizing the filtering by grass blades. Widths up to 12 feet may be used if separated by a dividing berm or structure to avoid braiding.

Manning's *n* value: The roughness coefficient, *n*, varies with the type of vegetative cover and flow depth. At very shallow depths, where the vegetation height is equal to or greater than the flow depth, the *n* value should be approximately 0.15. This value is appropriate for flow depths up to approximately 4 inches. For higher flow rates and flow depths, the *n* value decreases to a minimum of 0.03 for grass channels at a depth of approximately 12 inches. The *n* value must be adjusted for varying flow depths between 4" and 12" (see Figure 7.5 for variable *n* values with varying depths).

Side slopes: The sides slopes should be flat as possible to aid in providing pretreatment for lateral incoming flows and to maximize the channel filtering surface. Steeper side slopes are likely to have erosion gullying from incoming lateral flows. A maximum slope of 3:1 is recommended (33%), a 4:1 slope is preferred where space permits.

Channel longitudinal slope: The slope of the channel should be steep enough to ensure uniform flow and which can be constructed using conventional construction equipment without ponding, but not steeper than 4.0%. A minimum slope of 1.0% is recommended.

Flow depth: The maximum flow depth for water quality treatment should be approximately the same as the height of the grass. Since most channels will be mowed relatively infrequently the vegetation may reach heights of 6" or more. However, since higher grass is prone to fallover during higher flows, a maximum flow depth of 4" is required for water quality design. The flow depth for the 2 year and 10 year storms will depend on the flow rate and channel geometry.

Flow velocity: The maximum flow velocity for water quality treatment should be sufficiently low to provide adequate residence time within the channel. A maximum flow velocity of 1.0 feet per second for water quality treatment is required. The maximum flow velocity for the 2 year storm should be non-erosive (a rate of 4.0 to 5.0 feet per second is generally recommended). The permissible velocities of several grass species are listed in Appendix A. Velocity values are purely guidelines and may not always be representative of field conditions. The 10 year permissible velocity may be somewhat higher due to the low frequency of occurrence. A permissible maximum rate of approximately 7.0 feet per second for this event is recommended.

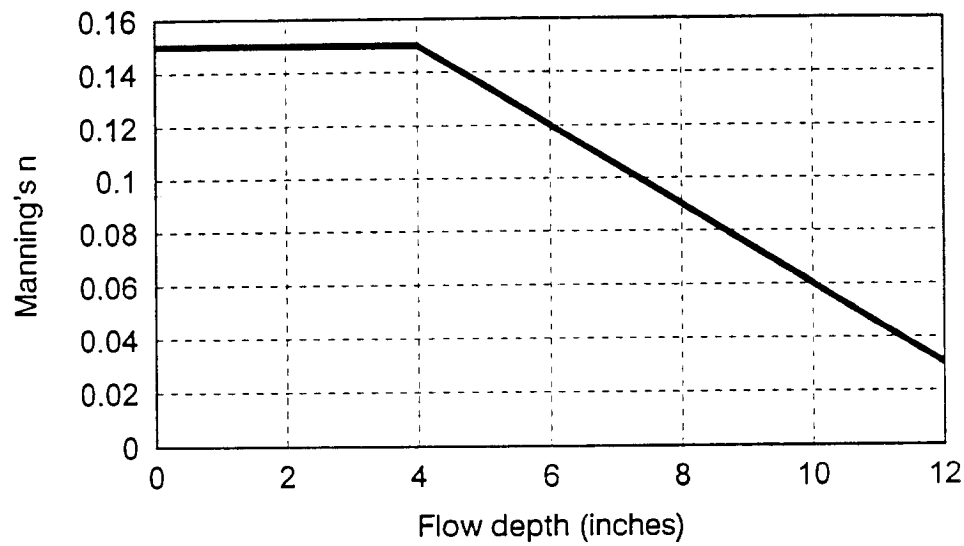
Length of channel: Generally grass channel length (for conveyance) is a function of site drainage constraints and a required length is not necessary. However, for water quality treatment, a minimum residence time of 10 minutes should be obtained to facilitate filtering. The minimum length required for water quality treatment grass channels is equal to the velocity, in feet per second multiplied by the minimum residence time of 600 seconds.

TABLE 7.2: DESIGN CRITERIA FOR TRAPEZOIDAL GRASSED CHANNELS FOR WATER QUALITY TREATMENT

Parameter	*Design Criteria*
Bottom Width	2 feet minimum, 6 feet maximum *
Side Slopes	3:1 or flatter
Channel Longitudinal Slope	1.0% minimum, 4.0% maximum
Flow Depth	4" for water quality treatment
Manning's *n* Value	0.15 for water quality treatment (depths ≤ 4") varies from 0.15 to 0.03 for depth between 4" and 12" 0.03 minimum for depths ≥ 12 inches (see Figure 7.5)
Flow Velocity	1.0 fps for water quality treatment 4.0 fps to 5.0 fps for 2 year storm 7.0 fps for 10 year storm
Length	Length necessary for 10 minute residence time

*Widths up to 12' are allowable with a dividing berm or structure.

FIGURE 7.5: VARIABLE MANNING'S N WITH VARYING FLOW DEPTH



GRASSED CHANNEL DESIGN PROCEDURE

- Use the 90% Rule to select rainfall for the Water Quality Storm (refer to Chapter 2, Section 2.5)
- Compute the peak rate of discharge (Q_p) for the Water Quality Storm based on the procedures identified in Chapter 2, Section 2.8, Small Storm Hydrology.
- Utilize Q_p to size the channel, maintain design criteria parameters noted in Table 7.2
  - Utilize the design charts (Figures 7.6-7.8) for channel widths 2, 4, and 6 feet, or
  - Utilize computer model which solves Manning's equation, or other open channel flow equations.
- Compute 2 year and 10 year frequency storm event peak discharges using SCS, TR-55.
- Check 2 year velocity for erosive potential (adjust geometry, if necessary and re-evaluate WQV design parameters).
- Check 10 year depth and velocity for capacity (adjust geometry, if necessary and re-evaluate WQV and 2 year design parameters).
- Provide minimum freeboard above 10 year storm water surface elevation (6 inches minimum, recommended).

The design charts provided (Figures 7.6 - 7.8) solve Manning's equation for various slopes and discharges. The charts were adapted from the U.S. Department of Transportation, Federal Highway Administration's, Hydraulic Design Series "Design Charts for Open-Channel Flow," reprinted 1980.

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EXAMPLE CALCULATION

Given: Roadway draining 1.0 acre drainage area
0.6 acre narrow paved surface
0.4 acre grassed pervious surface, silty soils

from Chapter 2, Small Storm Hydrology
R_v = [0.6(0.70) + 0.4(0.11)]/1.0 = 0.46
for 1.0" rainfall
WQV = 1.0"(0.46)*1.0=0.46"

Using modified TR-55 methodology
(from equation 2.3)
CN = 1000/[10 + 5(1.0) + 10(.46) - 10[(.46)^2 + 1.25(.46)(1.0)]^{½}]
CN = 93.2 use 93
for time of concentration (t_c) = 6 minutes, = 0.1 hour

and
I_a = 0.151, I_a/P = .151/1.0 = 0.15
q_u = 1000 csm/in
Qp = 1000 csm/in(1.0 ac/640ac/mi^2)(.46 in) = .72 cfs, use 0.7 cfs

See chart (Figure 7.7) for 4' bottom width
for *n* = 0.15, Q*n* = (0.7)(0.15) = 0.11, and 4' bottom width, read depth = .29, and V*n* = 0.08
for 2.0% slope, V = 0.08/0.15 = 0.5 fps
minimum length for 10 minute residence time, L = 0.5 ft/sec(600 sec) = 300 feet

Using traditional TR-55 methodology
for 2 and 10 year storm (2 year rainfall = 3.0 inch and 10 year rainfall = 5.0)
CN = [0.6(98) +0.4(61)]/1.0 = 83.2 use 83
t_c = 0.1 hour
I_a = .41, I_a/P = .41/3.0 = .14
volume of runoff, 2 year = 1.45 inches
10 year = 3.17 inches
q_u = 1000 csm/in
2 year Q_p = 1000 csm/in(1.0 ac/640ac/mi^2)(1.45 in) = 2.3 cfs
10 year Q_p = 1000 csm/in(1.0 ac/640ac/mi^2)(3.17 in) = 5.0 cfs

Figure 7.7: 2 year, use *n* =0.10, Q*n* = (2.3cfs)(0.1) = 0.23, slope = 2.0%; depth = .45 feet
and V*n* = .10, therefore V = 1.0 fps
10 year, use n = 0.08, Q*n* = (5.0cfs)(0.08) = 0.4, slope = 2.0%; depth = 0.6 feet
and V*n* = .12, therefore V = 1.5 fps

Use 4 foot wide channel, with 3:1 side slopes, 2.0% slope, with a minimum depth of 0.6 + 0.5 = 1.2 feet, note *n* is lower for the 2 and 10 year events as the depth increased (once the depth exceeds approximately 12 inches use *n* = 0.03).

FIGURE 7.6: DESIGN CHART FOR TRAPEZOIDAL CHANNEL (2' BOTTOM WIDTH)

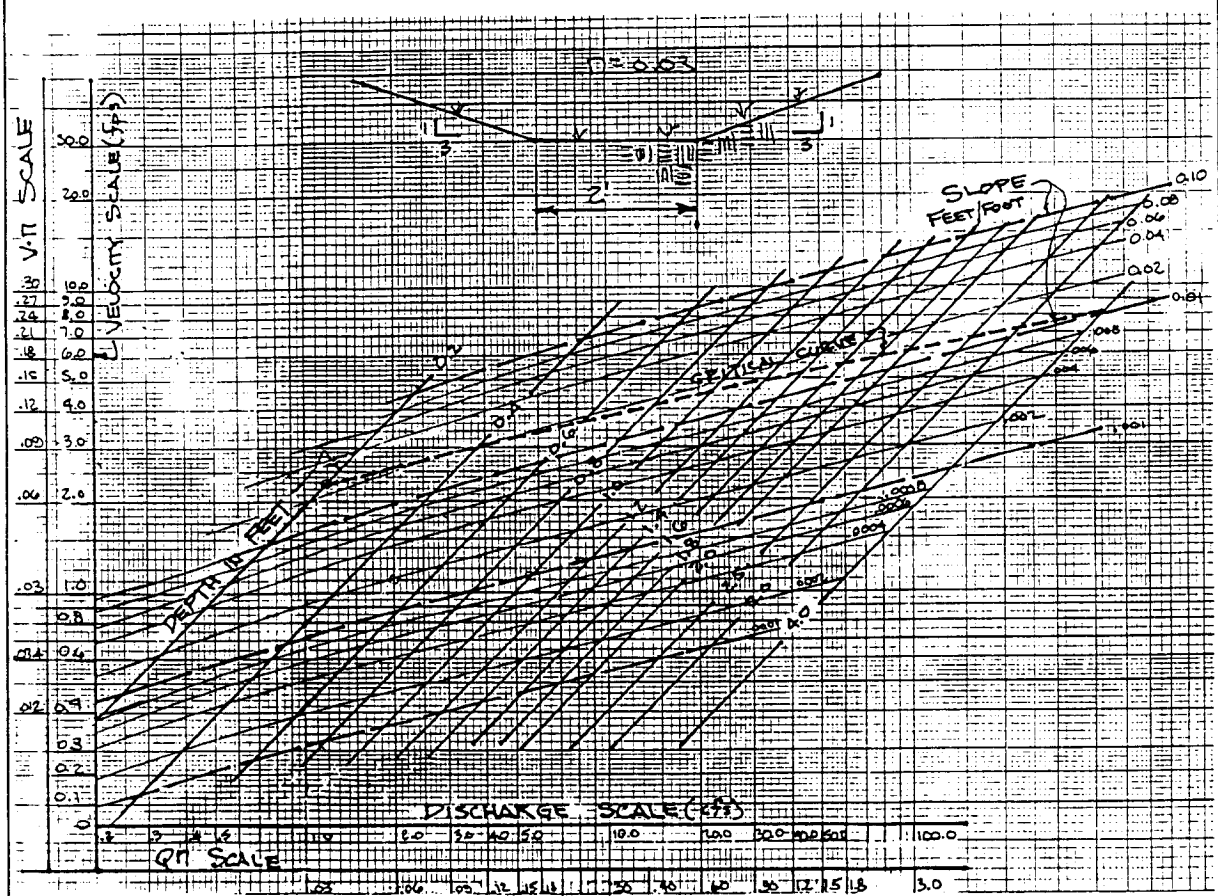


FIGURE 7.7: DESIGN CHART FOR TRAPEZOIDAL CHANNEL (4' BOTTOM WIDTH)

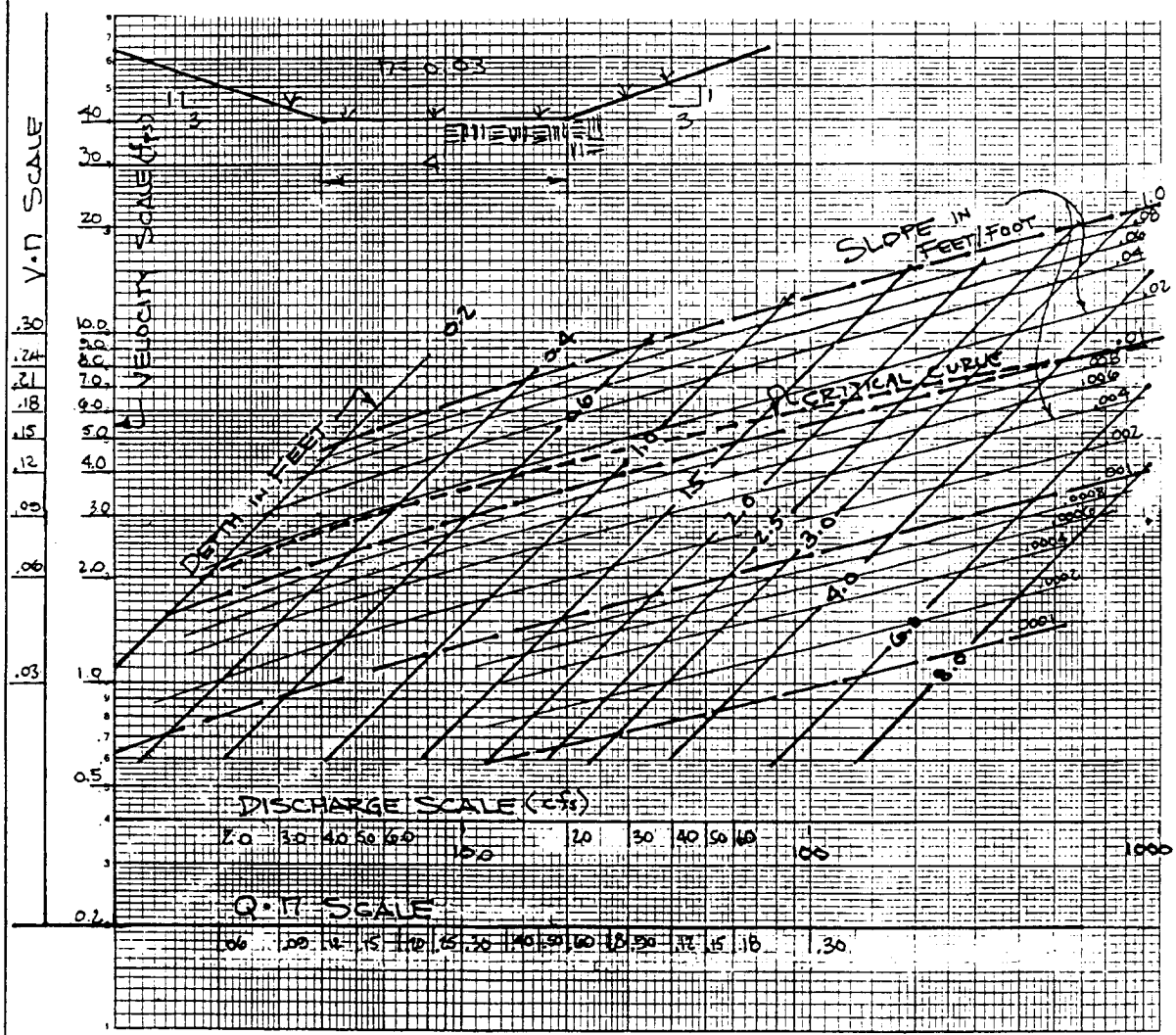
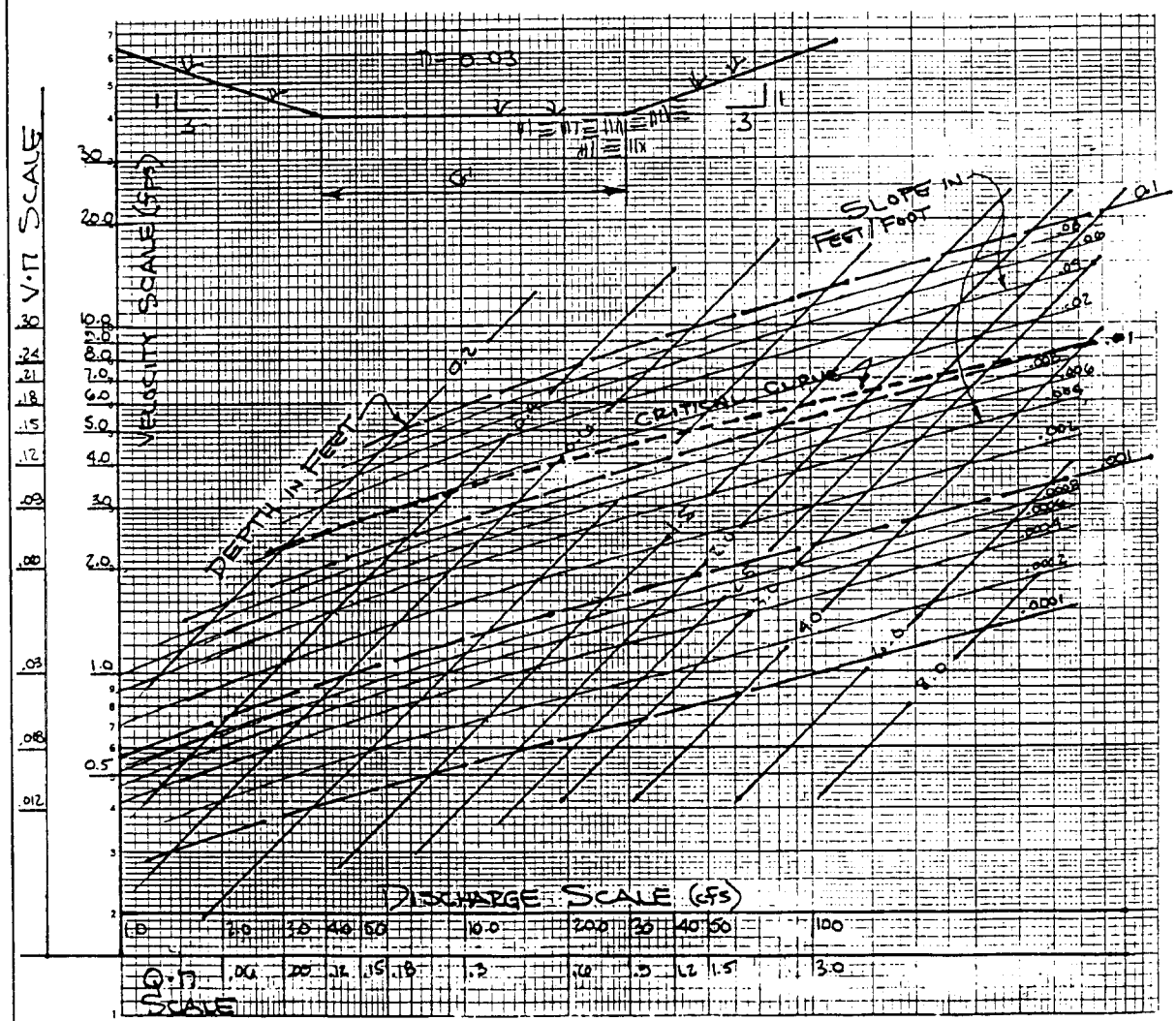


FIGURE 7.8: DESIGN CHART FOR TRAPEZOIDAL CHANNEL (6' BOTTOM WIDTH)



DRY SWALE AND WET SWALE DESIGN CONSIDERATIONS

The design approach for sizing dry and wet swales is based on temporarily storing the WQV within a shallow ponding area. This methodology incorporates a **volume based** sizing criteria for the WQV, and a rate based criteria for checking the erosive potential for the 2 year frequency storm and capacity for the 10 year frequency storm.

The dry swale is mainly applied to moderate to large lot residential land uses. Small impervious areas (small parking lots and rooftops) and rural highway runoff can be accommodated by the dry swale. Wet swales are predominately used for highway runoff applications, but can also be used to filter water from small parking lots, rooftops and pervious areas (see Chapter 3).

The specific design considerations are presented below, and summarized in Table 7.3.

Shape: The swales should generally be trapezoidal in shape, although a parabolic shape is also acceptable (provided the underlying soil bed design width, for dry swales, is equal to or greater than, the design bottom width for a trapezoidal cross section). The criteria presented in this section assumes a trapezoidal cross section.

Bottom width: For the trapezoidal cross section, size the bottom width between two and eight feet. The two feet minimum allows for construction considerations and ensures a minimum filtering surface for water quality treatment. The eight feet maximum reduces the likelihood of flow channelization within a portion of the bottom of the swale. Eight feet is allowed (versus the six feet specified for grass channels) to accommodate additional storage for WQV. Widths up to 16 feet may be used if separated by a dividing berm or structure to avoid braiding.

Side slopes: The side slopes of the channel should be no steeper than 2:1 for maintenance considerations (mowing). Flatter slopes are encouraged where adequate space is available to aid in providing pretreatment for lateral flows. The steeper maximum side slope for the dry/wet swales is permitted because these practices are designed to retain a storage volume versus being designed for a minimum residence time.

Swale longitudinal slope: The slope of the swale should be moderately flat to permit the temporary ponding of the WQV within the channel without having excessively deep water at the downstream end. A slope between 1.0% and 2.0% is recommended. When natural topography necessitates, steeper slopes may be acceptable if check dams (vertical drops of 6 to 12 inches) are used. These structures will require additional energy dissipating measures and should be placed no closer than 50 to 100 feet intervals (see Figure 7.3).

TABLE 7.3: DESIGN CRITERIA FOR DRY AND WET SWALE SYSTEMS

Parameter	*Swale Design Criteria*
Pretreatment volume	.05" per impervious acre, at initial inflow point.
Preferred shape	Trapezoidal or parabolic.
Bottom width	2 feet minimum, 8 feet maximum widths up to 16 feet are allowable if a dividing berm or structure is used.
Side slopes	2:1 maximum, 3:1, or flatter preferred.
Longitudinal slope	1.0% to 2.0% without, check dams.
Sizing criteria	Length, width, depth, and slope needed to provide surface storage for WQV. Outlet structures sized to release WQV over 24 hours.
Underlying soil bed	Equal to swale width Dry Swale: Moderately permeable soils (USCS ML, SM, or SC) 30" deep with gravel/pipe underdrain system Wet Swale: Undisturbed soils, No underdrain system
Depth and capacity	▸ Surface storage of WQV with a maximum ▸ depth of 18 inches for water quality treatment (12" average depth). ▸ Safely convey 2 year storm with non-erosive velocity (≤ 4.0 to 5.0 ft/s) ▸ Adequate capacity for 10 year storm with 6" of freeboard

Design sizing criteria: The detention/retention capacity of both dry and wet swales is governed by the runoff associated with the "water quality storm." The swale length, width, depth, and slope should be designed to temporarily accommodate the WQV through surface ponding. For the dry swale, all of the surface ponding should dissipate within a maximum 24 hour duration. The outlet structure (half-round pipe in Figure 7.2) is sized to release the WQV over 6 hours. Using perforations in the bottom 6" of pipe. The soil media will have an infiltration capacity of at least a foot/day.

For wet swales, the WQV volume is still retained for 24 hours, but ponding may continue indefinitely depending on the depth and elevation to the water table. The WQV for high density residential, commercial and industrial land uses will most likely be too great to be accommodated with most swale designs. However, swales may be appropriate for pretreatment in association with other practices for these higher density land uses or may be acceptable solutions for watershed retrofit projects (see Chapter 3).

Underlying soil bed: The soil bed below the dry swale should consist of a moderately permeable soil material with a high level of organic matter (USCS ML, SM, or SC). The soil bed should be 30 inches deep, and should be accompanied by a gravel/pipe underdrain system. This soil mixture is necessary in residential areas to ensure drainage of the swale system within a moderately short time period to avoid safety and nuisance concerns. The soil/gravel interface should be roto-tilled to have an approximate six inch mixing zone of fine sand, soil, and gravel to augment the filtering capability of the practice.

The soil bed below the wet swale should consist of undisturbed soils. This area may be periodically inundated and remain wet for long periods of time, and is therefore not appropriate for residential land uses. (Filter fabric is not recommended for this section).

Swale depth and capacity: Swales should be designed to provide a shallow ponding depth for the WQV (a maximum depth of 18" for the WQV is recommended), safely convey the 2 year storm with design velocities less than 4.0 to 5.0 feet per second, and provide adequate capacity for the 10 year storm with a minimum of 6 inches of freeboard.

DRY SWALE AND WET SWALE DESIGN PROCEDURE

- Use the 90% Rule to select rainfall for the Water Quality Storm (refer to Chapter 2, Section 2.5).
- Compute the Water Quality Treatment Volume (WQV) for the given land surfaces, based on the procedures identified in Chapter 2, Section 2.7, Small Storm Hydrology.
- Identify the required swale bottom width, depth, length, and slope necessary to store the WQV within a shallow ponding depth (≤ a maximum depth of 18").
- Compute the WQV drawdown time to ensure that it is less than 24 hours.

- Compute the 2 year and 10 year frequency storm event peak discharges using SCS, TR-55.
- Check the 2 year velocity for erosive potential (adjust swale geometry, if necessary and re-evaluate WQV design parameters.
- Check the 10 year depth and velocity for capacity (adjust swale geometry, if necessary and re-evaluate WQV and 2 year design parameters).
- Provide minimum freeboard above 10 year storm water surface profile (6 inches minimum recommended).
- Specify vegetation required to meet design conditions (see Appendix B).
- For dry swales, specify grasses resistant to periodic inundation and periodic drought.
- For wet swales, specify grasses resistant to sustained inundation and/or water table at or near the surface, wetland species are appropriate for swale bottom.
- For all swales, check permissible velocities of selected vegetation to ensure the 2 year frequency storm velocity is non-erosive.

EXAMPLE CALCULATION

Given: Four, approximately ⅓ acre lots
4 driveways and a 26 foot wide open section road
A 1.55 acre total drainage area
(See Figure 7.9)

Houses = 0.22 acres
Driveways = 0.10 acres
Street = 0.15 acres
Pervious surface (lawns) = 1.08 acres

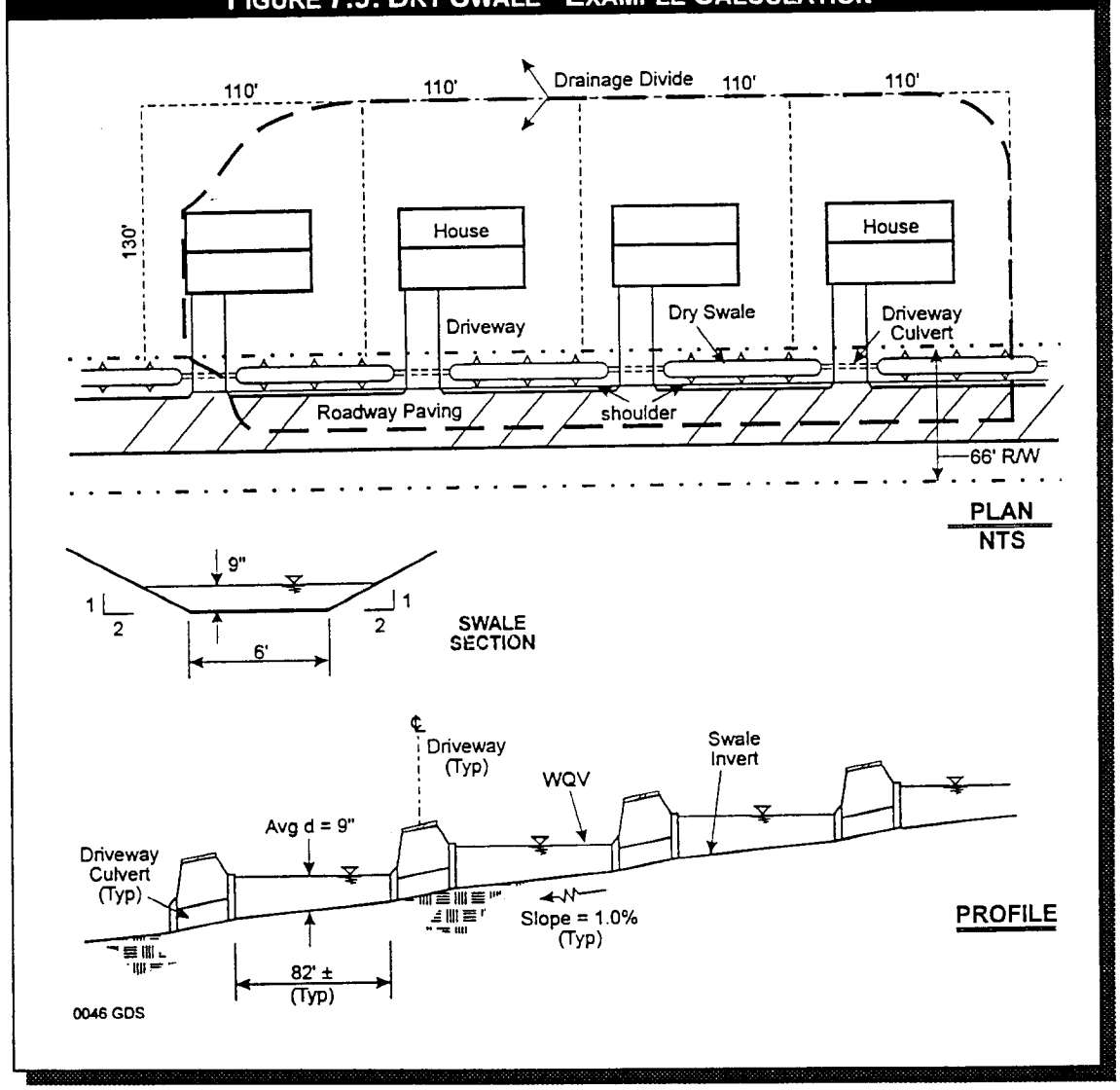
from Chapter 2, Small Storm Hydrology
R_v = .97(0.22) + .70(0.25) + .11(1.08) / 1.55 = 0.33
for 1.0" rainfall
WQV = 1.0"(0.33) = 0.33"
WQV = 0.33" * 1 ft/12" * 1.55 ac * 43,560 ft^2 / ac = 1,857 ft^3

Size dry swale to provide minimum storage = 1,857 ft^3
from Figure 7.9, length available = 328 ft (4 * 82')

try 6 ft bottom width, 9" depth, 2:1 side slopes swale
A = (6.0)(.75) + 2 * ½(1.5)(.75) = 5.63 ft^2
Vol. provided = 328 ft * 5.6 ft^2 = 1,845 ft^3 which is ≈ 1,857 ft^3

Note: Swale must have a minimum 1.0% slope, and must have approximately 82 feet separating each driveway, so as to maintain an average 9" depth, set depth = 9" ft at ½ distance between driveways, therefore, use a 14" maximum depth at culvert inlets and 4" at culvert outlets.

FIGURE 7.9: DRY SWALE - EXAMPLE CALCULATION



FILTER STRIP DESIGN CONSIDERATIONS

The design approach for sizing filter strips is based on temporarily storing the WQV within a shallow ponding area. This methodology also utilizes a volume based sizing criteria for the WQV.

This practice is primarily designed for pervious surfaces and rooftops (rear yard runoff from single and multi-family residential). Filter strips may also be appropriate

for some small parking lots and residential land uses where adequate treatment space is available (see Chapter 3).

The specific design considerations are presented below and summarized in Table 7.4.

TABLE 7.4: DESIGN CRITERIA FOR FILTER STRIPS

Parameter	*Filter Strip Design Criteria*
Sizing criteria	Length, depth and slope necessary to provide surface storage for WQV. Width equal to area draining to filter. Minimum length = 25 feet
Slope	Minimum slope = 2.0% Maximum slope = 6.0%
Treatment drainage area	Maximum overland flow lengths: pervious surfaces = 150 feet impervious surfaces = 75 feet

Size: The size of the filter strip is determined by the required treatment volume, however the minimum length must be 25 feet. The width of the filter strip is equal to the width of the area draining to it.

Slope: The maximum slope should be no more than 6.0% and the minimum slope should be no less than 2.0%. Steeper slopes will increase velocities and lead to concentration of runoff and likelihood for erosion. In addition, as the slope increases, the treatment volume per cross sectional area decreases. Slopes flatter than 2.0% may be appropriate for some geographic regions, but are discouraged in residential areas due to the tendency for surface ponding to create potential nuisance conditions.

Drainage Area: The maximum drainage area to filter strips is limited by the overland flow limits of 150 feet for pervious surfaces and 75 feet for impervious surfaces.

FILTER STRIP DESIGN PROCEDURE

- Prepare site grading design to limit length of overland flow entering filter strip.
- Design filter strip to receive runoff through sheet flow along a level plain. For paved surfaces, provide multiple entry points (utilizing curb stops, instead of concrete curb and gutter). For pervious surfaces, ensure that the entry point is level at the top of the filter strip. A concrete "level spreader" may be necessary for some applications.
- Compute the WQV using the 90% Rule (refer to Chapter 2, Section 2.5 and Section 2.7).
- Check available storage within shallow ponding limits, based on slope, height of pervious berm and width of filter strip.
- Size outlet pipes to ensure release of the WQV drawdown time is less than 24 hours.
- Layout filter strip to maintain 25 feet minimum length.
- Maintain the toe of the filter strip as level as possible.
- Construct the filter strip outside of the boundary of a natural stream buffer area.
- Provide overflow or bypass for storms larger than WQV (e.g., 2 and 10 year events).
- Specify vegetation required to meet design conditions (see Appendix B):

  - Specify grasses resistant to frequent inundation within shallow ponding limit.

  - Specify grasses with high retardance or high permissible velocities for sloping area leading to shallow ponding area.

EXAMPLE CALCULATION

Single family lot, 150' deep x 100' wide
Drainage area = 15,000 sq. ft. or 0.34 acres
Single family house (60' x 40') = 0.06 ac
Pervious surface (lawn) = 0.28 ac

From Chapter 2, Small Storm Hydrology
R_v = [.97(.06) + .11(.28)]/.34 = 0.26
for 1.0" rainfall
WQV = 1.0"(0.26) = 0.26"
WQV = 0.26" 1 ft/12 in * 15,000 sq. ft. = 325 ft^3

Assume filter strip slope = 2.0%
Pervious berm height = 6"
for 25' minimum length
cross sectional area = ½ * 25' * 0.5' = 6.25 ft^2

Required filter strip width: = 325 ft^3/6.25 ft^2 = 52 ft
100 foot width provided, volume acceptable

Use 25' long x 100' wide filter strip with 6" high pervious berm

7.6 OVERFLOW

The overflow element of the four vegetated filter design variations consist of safely conveying the high flow events (storms greater than the WQV) through the systems. In the case of the grass channel, dry swale, and wet swale, this involves ensuring that the velocities of more frequent high flow events (e.g., 2 year storm) are non-erosive, and that the less frequent high flow events (e.g., 10 year, or in some cases, the 100 year storm) are contained within the channel, and do not pose a flooding condition to adjacent areas. In the case of the filter strip, this involves either diverting the higher flows to by-pass the practice, or providing an overflow spillway to ensure a non-erosive condition.

7.7 MATERIAL SPECIFICATIONS

Table 7.5 identifies many of the material specifications necessary for the design of the four vegetated filter design variations. Specific information on the use of various grasses is presented in Appendix A. Detailed seeding specifications are not included. Designers should refer to their local landscaping specifications.

TABLE 7.5: MATERIAL SPECIFICATIONS

Parameter	Specification	Size	Notes
Dry Swale Soil	Sand: ASTM C-33 fine aggregate concrete sand Soil: USCS: ML, SM, or SC	Sand: .02" - .04"	Soil with a higher percent organic matter is preferred
Check Dam	Pressure treated or equiv. rot resistant wood (e.g., black locust)	6" x 6"	Embed 2-3' into side slopes
Filter Strip Sand/Gravel Pervious Berm	Sand: ASTM C-33 fine aggregate concrete sand Gravel: AASHTO M-43	Sand: .02" - .04" Gravel: ½" - 1"	Mix with approx. 25% loam soil to support grass cover crop
Pea Gravel Diaphragm and Curtain Drain	ASTM D 448 size no. 6	Varies (approximately ⅛" - ⅜")	Use clean bank-run river pea gravel
Underdrain Gravel	AASHTO M-43	½" - 2"	
PVC Piping	AASHTO M-278	6" - Rigid Schedule 40	⅜" perf. @ 6" centers, 4 holes per row

7.8 MAINTENANCE GUIDELINES

The following general maintenance elements are applicable to all of the four vegetative practice design variations. Inspections are an integral part of any maintenance program. These four vegetative filtering practice variations should be inspected on a semi-annual basis for the first year, and after major storm events. After the first year, annual inspections should be sufficient.

7.8A PRETREATMENT

- The pea gravel diaphragm should be inspected annually for clogging. Sediment build-up should be removed, as needed. Replacement of the diaphragm may be warranted when the voids are obviously filled with sediment and water no longer percolates into the stone.

- The grass vegetation along the side slopes should be inspected for erosion rills or gullys, and corrected, as needed. Bare areas should be seeded, or sodded, as necessary.

- The initial inflow forebay should be inspected annually for sediment build-up. Any excessive sediment, trash, and debris should be removed and disposed of in an appropriate location.

7.8B DRY SWALE SAND AND SOIL BED

- The sand/soil bed may experience some erosion, particularly at the inflow point. Periodic inspection and correction of erosion areas may be necessary.

- The surface of the bed may become clogged with fine sediments over time. If the swale does not drain within 48 hours, roto-tilling, or cultivation of the top of the soil bed may be required to ensure adequate filtration.

7.8C VEGETATION

- Grass should be mowed on a regular basis. What is considered a regular basis will depend on the location, the type of practice (e.g., wet or dry), climate, and type of grass selected. In general, in order to maintain the optimal filtering capability, grass levels should not exceed 3 to 4 inches. This may require mowing as frequently as bi-weekly during the peak growing season. Wet swales, which incorporate wetland vegetation, do not require mowing at the same frequency as the other practices.

- The grasses should be selected based on the hydric conditions anticipated (refer to Appendix B). However, water table conditions vary from season to season, and from location to location, and the specified grass may not establish itself sufficiently. Annual inspection of the vegetation condition is necessary. An alternative grass species should be considered for species which fail to establish.

- Sediment build-up within the bottom or the channel, swale or filter strip should be removed when it has accumulated to approximately 25% of the original design volume or channel capacity.

Appendix A

Stormwater Pollutant Concentrations from Different Urban Source Areas and Hotspots

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APPENDIX A.

STORMWATER POLLUTANT CONCENTRATIONS FROM DIFFERENT URBAN SOURCE AREAS AND HOTSPOTS

Mean concentrations in the table are unweighted averages of published mean concentrations in the indicated reference. Some authors had multiple sites of an individual source area or hotspot.

Source	*N*	*References*
Resid Roof	3	Bannerman (1994), Pitt and McLean (1986)
Comm Roof	2	Bannerman (1994), Pitt and McLean (1986)
Indust Roof	5	Pitt et al. (1994), Good (1993), Bannerman (1994), Thomas and Greene (1993)
C/R Parking	9	Bannerman, Rabinal, Pitt et al., Pitt and McLean, Bell, Schueler
Ind Parking	4	Bannerman (1994), Pitt et al. (1994) Horner (1994)
Res Street	4	Bannerman (1994), Pitt and McLean (1986), Schueler and Shepp (1993)
Comm Street	4	Bannerman (1994), Pitt et al. (1994)
Rural Highway	6	FHWA (1990)
Urban Highway	8	FHWA (1990)
Lawns	2	Bannerman (1994), Pitt et al. (1994)
Driveway	1	Bannerman (1994)
Gas Stat/VMA	3	Pitt et al. (1994), Schueler (1994), Rabanal & Grizzard (1995)
Auto Recycler	2	Swamikannu (1994)
Heavy Indus	1	Leersnyder (1993)
Landscaping	1	Pitt et al. (1994)
NURP-DC	-	MWCOG (1983)
NURP-US		USEPA (1983)

NOTES: NURP US lead value is considered unrepresentative as it reflects deposition conditions of leaded gas.

STORMWATER POLLUTANT CONCENTRATIONS FROM DIFFERENT URBAN SOURCE AREAS AND HOTSPOTS

Shaded cells indicate mean concentrations twice the national or regional average.

Source	N	TSS	F Coli	Cd	Cu	Pb	Zn	PP	O/G
Resid Roof	3	19	.26	3	20	21	312	7	NA
Comm Roof	2	9	1.1	0.3	7	17	256	16	NA
Indust Roof	5	17	5.8	2	62	43	1390	6	NA
C/R Parking	9	27	1.8	8	51	28	139	13	8.5
Ind Parking	4	228	2.7	2	34	85	224	0	15
Res Street	4	172	37	1	25	51	173	1	2
Comm Street	4	468	12	6.7	73	170	450	13	NA
Rural Highway	6	41	NA	NA	22	80	80	NA	NA
Urban Highway	8	142	NA	1	54	400	329	NA	NA
Lawns	2	602	24	ND	17	17	50	NA	NA
Driveway	1	173	34	0.5	17	ND	107	NA	NA
Gas Stat/VMA	3	31	NA	9	88	80	290	19	14
Auto Recycler	2	335	NA	8.5	103	182	520	NA	25
Heavy Indus	1	124	NA	NA	148	290	1600	NA	NA
Landscaping	1	37	NA	0.3	94	29	263	14	NA
NURP-DC	-	100	NA	0.5	10	18	37	2	NM
NURP US		150	NA	0.7	34	140***	160	3	3

NOTES: TSS (mg/l), F. coliforms (1000 counts/ml), Cd, Cu, Pb and Zn (ug/l), PP= number of non-metal and non-pesticide priority pollutants detected in stormwater. O/G= oil and grease (mg/l), N= number of monitoring studies used to characterize a source area. Values are an un-weighted mean of each monitoring study. Total number of studies for a given parameter may be less than N.

Appendix B

Grass Species Selection Guide

Grass Type	Availability	Salt Tolerance	Resistance to Frequent Inundation	Permissible Velocity ft/s	Native or Introduced	Growth Season /	Germination Rate / Growth Rate	Drought Resistance	Bunch / Sod	Root Network Density	Height at Maturity	Optimal Soil Conditions	Regional Adaptation	Shade Tolerance	Notes:
Bermuda grass *(Cynodon dactylon)*	Yes	Good	Excellent	6	N	Warm	Rapid, highest growth rate of warm season grasses	Good	Sod	Rhizomes form tight sod, fibrous extensive roots	.75-2 ft.	Fine textured, fertile, well drained soil, pH 5.5-7.5	Southeast, most of southern US with irrigation	Low	Good tolerance to heat. Poor tolerance to cold. Many varieties grown from sprigs
Bahiagrass *(Paspalum notatum)*	Yes	Good	Good	5	I	Warm	Slow establishment	Fair		Rhizomatous	1.5 ft.	Droughty, sandy ,fine, poorly drained, slightly acidic soils	Southeast	Good	Germination enhanced by scarification. Intolerance to low temperatures
Big Bluestem *(Andropogon gerardii)*	Yes	Fair	Fair		N	Warm	Slow est.	Fair	Sod	Dense sod	4-6 ft.	Silt-clay	Midwest, Great Plains, with irrigation.	Low	Low maintenance, low nitrogen requirement. Seeded in "warm-season" mixtures
Creeping Bent*grass* *(Agrostis palustris)*	Yes	Good	Excellent		I	Cool		Poor	sod	Dense fibrous, medium to shallow in depth, vigorous stolons	1-12"	Best adapted to fertile, fine textured, slightly acidic soils. pH5.5-6.5	Northeast, Northern Great Plains, Midwest	Fair	Hardy cool season turf grasses. Tolerent to low temperature extremes
Buffalograss *(Buchloe dactyloides)*	Yes	Fair	Excellent	5	N	Warm	Fast est.	Excellent	Bunch	Shallow, good soil and sand binder	.33-1 ft.	Fine textured soils. Tolerates alkaline soils, silt-clay	Great Plains	Low	Excellent hot temp. hardiness. Seed only in mixtures. Seeded by stolons
Chewings Fescue (*Festuca rubra* var. *Commutata)*			Good			Cool	Good establishment rate	Excellent	Bunch			Acidic, infertile, sandy soils	Northern Great Plains	Good	Tolerant to cold temperatures, does well in full sun, may have endophytes
Russian wild rye *(Elymus junceas)*					I	Cool		Good	Bunch		13-24"				

GRASS SPECIES SELECTION GUIDE

Grass Type	Availability	Salt Tolerance	Resistance to Frequent Inundation	Permissible Velocity ft/s	Native or Introduced	Growth Season /	Germination Rate / Growth Rate	Drought Resistance	Bunch / Sod	Root Network Density	Height at Maturity	Optimal Soil Conditions	Regional Adaptation	Shade Tolerance	Notes:
Crownvetch *(Coronilla varia)*	Yes	Fair	Low		I		Slow germination. Rapid growth	Good		Deep		Well drained, pH 6.5-7.0	Northeast, Midwest, Great Plains		Requires Liming
Standard Crested Wheatgrass *(Agropyron desertorium)*	Yes	Fair	Poor		I	Cool	Easily established, slower Growth	Excellant	Bunch	Extremely fibrous, deep	12-30"	Fertile soils from clayey to a sandy loam	Pacific Coast, Southwast, Northern Great Plains		Poorly adapted to hot temperures. Seeded alone
Hard Fescue *(Festuca ovina* var. *duriuscula)*	Yes	Poor	Poor		N	Cool	Fair to estabilish, good growth	Fair	Bunch		12-24"		Pacific Coast, Northern Great Plains	Good	Used frequently in erosion control
Kentucky Bluegrass *(Poa pratensis)*	Yes	Poor	Fair		5 I	Cool	Good est. rate, good growth rate	Fair	Sod	Dense	12-25"	Moist, well drained, fertile, medium texture, pH 6-7	Midwest, Northeast	Fair	Medium hot temp. tolerance, but subject to summer dormancy. Good cold Temperture hardiness
Red Fescue *(Festuca rubra)*	Yes	Poor	Poor		2.5 I	Cool		Good	Sod		12-30"				Fair heat and cold tolerance
Reed Canarygrass *(Phalaris arundinacea)*	Yes	Poor	Excellent		5 N-I	Cool	Harder to establish, good growth rate	Fair	Sod	Shallow, rhizomatous	5'	Silty, clayey soils	Northeast, Southeast and Midwest, other areas with irrigation	Poor	High wildlife value. Not recomended for foot traffic. Seeded or spread by sod

GRASS SPECIES SELECTION GUIDE

Grass Type	Availability	Salt Tolerance	Resistance to Frequent Inundation	Permissible Velocity ft/s	Native or Introduced	Growth Season /	Germination Rate / Growth Rate	Drought Resistance	Bunch / Sod	Root Network Density	Height at Maturity	Optimal Soil Conditions	Regional Adaptation	Shade Tolerance	Notes:
Red Top *(Agrostis alba)*	Yes	Fair	Good	2.5	I	Cool	Fair establishment, good growth rate	Poor	Loose sod	Short rhizomes do not form tight soils	25"	Wet. poorly drained, low fertility, acidic soils. ph 5.5 - 5.6	Northeast, Midwest, other areas with, irrigation	Good	Fair heat tolerance, good cold temperature tolerance
Ryegrass Perennial *(Lolium perenne)*	Yes	Fair	Fair		I	Cool	Excellent establishment, fair growth rate	poor	Bunch		13-25"	Sandy, silty soils. pH 6 - 7.5	Midwest, Pacific coast, other areas with irrigation	Poor to fair	Poor to fair hot temperatue tolerance. Poor cold temperature hardiness
Annual rye *(Lolium multiflorum)*	Yes	Fair	Good		I	Cool, short lived	Highest germination rate of Cool season grasses	Poor			25"	Neutral to slightly acidic, medium to good fertility	Most of northern US	Fair to good	Tolerates wet soils, good nurse crop
Side Oats Grama` *(Bouteloua curripendula)*		Fair	Poor		N	Warm	Fast est.	Inferior to Blue Grama, but tolerant	Bunch		2-3 ft.	Variety of soils, provided adequate moisture is available		Poor	Blue Grama preferred over Side Oat in soil stabilization applications
Smooth Brome *(Bromus inermis)*	Yes	Fair	Fair		I	Cool	Fast est.	Fair	Sod	Extensive, rhizomatous	3-4 ft.	Deep well drained, fertile, fine texture, upland soils. Silts-clays	Northeast, Southeast, Midwest, other areas with irrigation	Good	Does well in sandy coarser texture soils also, if supplied w/ N
St. Augustine		Good			N		Fair to good	Fair	Sod			Moist organic well drained soils, pH 6.5		Good	Excellent hot temperture Hardiness, poor cold tolerance
Switchgrass *(Panicum virgatum)*	Yes	Fair	Good		N	Warm	Fast establisment, good growth rate	Fair	Sod	Shallow, rhizomatous	4-5 ft.	Lowland and upland, sandy and silty-clay soils	Southeast, Midwest, and Southern Great Plains	Poor	Hot temp. tolerant. Usually seeded with other warm-season grasses

GRASS SPECIES SELECTION GUIDE

Grass Type	Availability	Salt Tolerance	Resistance to Frequent Inundation	Permissible Velocity ft/s	Native or Introduced	Growth Season /	Germination Rate / Growth Rate	Drought Resistance	Bunch / Sod	Root Network Density	Height at Maturity	Optimal Soil Conditions	Regional Adaptation	Shade Tolerance	Notes:
Tall Fescue *(Festuca arundinacea0*	Yes	Good	Excellent	5	I	Cool	Fast establishment rate, good growth rate	Excellent, among the highest of the turf grasses	Bunch	Extensive, coarse, deep	3-4 ft.	Tolerates low fertility, pH 5.7 Silts-clays preferred	Northeast, Southeast, Midwest, other areas with irrigation	Fair	Doesn't blend well with other cool season turfs. Usually seeded in pure stands. Good hot temp. tolerance, fair cold hardiness
Timotyhy *(Phleum pratense)*	Yes	Poor	Poor		I	Cool	Fast est.	Fair	Bunch			Highly fertile, moist, fine grained soils. pH 6-7	Northeast, Midwest, Pacific Coast		Cold temperature hardy. Slow recovery from hot temperatures and water stress. Seeded in mixtures
Weeping Lovegrass *(Eragrostis curvula)*		Fair	Fair		I	Warm	Fast est	Fair	Bunch	Fibrous	12-24" 3-4 ft.	Upland, sandy soils	Southwest, Southern Great Plains		Seeded in pure stands
Western Wheatgrass *(Andropyron spicatum)*	Yes	Good	Good		N	Cool	Slow est. Good growth rate	Fair to good	Tight Heavy Sod	Fibrous roots and strong creeping rhizomes	2-3 ft.	Fine textured soils, silts-clays	Midwest through most of western US		Cold temp. hardy. Seeded in mixtures or pure stands. Tolerates silting
Ladino White Clover *(Trifolium repens)*	Yes	Poor	Fair to good		I	Cool, humid	Fast est. Fair growth rate	Poor		Shallow roots		Moist, fine textured soils	Northeast, Southeast, Midwest, other locations with irrigation		Creeping by stolons
Zoysiagrass	Yes	Good	Poor		I	Warm	Slow est.	Excellent	Sod	Fibrous roots and strong creeping rhizomes		Well drained, fine textured, fertile soils. pH 6-7		Good	Japanese variety is the most cold temp. tolerant with most rapid growth

GRASS SPECIES SELECTION GUIDE

Grass Type	Availability	Salt Tolerance	Resistance to Frequent Inundation	Permissible Velocity ft/s	Native or Introduced	Growth Season /	Germination Rate / Growth Rate	Drought Resistance	Bunch / Sod	Root Network Density	Height at Maturity	Optimal Soil Conditions	Regional Adaptation	Shade Tolerance	Notes:
Blue Grama *(Bouteloua gracilis)*	Yes	Fair	Fair		N	Warm	Slow est. Good growth rate	Excellent	Sod	Dense fibrous extensive, but shallow	1-2 ft.	Fine textured, upland Soils. Tolerates alkaline soils, silts-clays	Midwest and most of western US		Good cold temp. tolerance. Seeded in mixtures with other warm-season grasses
Orchardgrass *(Dactylis glomerata)*	Yes	Poor	Poor		I	Cool - Warm	Fast Est. Fair growth rate	Poor to fair	Sod			Silty and clayey soils	Northeast, Southeast, Midwest, most other areas with irrigation	Good	Seeded in mixtures

Appendix C

Bioretention Species Selection Guide

Appendix C Recommended Plant Species For Use In Bioretention - Herbaceous Species

Species	Moisture Regime		Tolerance						Morphology			General Characteristics		Comments
Scientific Name Common Name	Indicator Status	Habitat	Ponding (days)	Salt	Oil/ Grease	Metals	Insects/ Disease	Exposure	Form	Height	Root System	Native	Wildlife	
Agrostis alba redtop	FAC	Mesic-Xeric	1-2	H	_	H	H	Shade	Grass	2-3'	Fiberous Shallow	Yes	High	_
Andropogon gerardi bluejoint	FAC	Dry Mesic-Mesic	1-2	_	_	_	_	Sun	Grass	2-3'	Fiberous Shallow	Yes	High	_
Andropogon virginicus broomsedge	_	Wet meadow	1-2	L	_			Full sun	Grass	1-3'		Yes	High	Tolerant of fluctuating water levels and drought.
Carex vulpinoidea fox sedge	OBL	Freshwater marsh	2-4	L	_			Sun to partial sun	Grass	2-3.5'	Rhizome	Yes	High	_
Chelone glabra														
Deschampsia caespitosa tufted hairgrass	FACW	Mesic to wet Mesic	2-4	H	_	H	H	Sun	Grass	2-3'	Fiberous Shallow	Yes	High	May become invasive.
Glyceria striata fowl mannagrass, nerved mannagrass	OBL	Freshwater marsh, seeps	1-2	L	_			Partial shade to full shade	Grass	2-4'	Rhizome	Yes	High	_
Hedera helix English Ivy	FACU	Mesic	1-2	_	_	_	H	Sun	Evergreen ground cover	_	Fiberous Shallow	No	Low	_
Hibiscus palustris														
Iris kaempferi														
Iris pseudacorus yellow water iris	OBL	Mesic to wet Mesic	2-4	L	_	H	H	Sun	Thin broad leaves	1-4'	Bulb	Yes	Med	_

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OBL Obligate Wetland - Occur almost always in wetlands

Adapted from the Prince George's County Design Manual for use of Bioretention in Stormwater Management

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Appendix C Recommended Plant Species For Use In Bioretention - Herbaceous Species

Species	Moisture Regime		Tolerance						Morphology			General Characteristics		Comments
Scientific Name **Common Name**	**Indicator Status**	**Habitat**	**Ponding (days)**	**Salt**	**Oil/ Grease**	**Metals**	**Insects/ Disease**	**Exposure**	**Form**	**Height**	**Root System**	**Native**	**Wildlife**	
Lobelia siphilitica														
Lotus Corniculatus birdsfoot-trefoil	FAC	Mesic-Xeric	1-2	H	L	H	H	Sun	Grass	2-3'	Fiberous Shallow	Yes	High	Member of the legume family.
Onoclea sensibilis sensitive fern, beadfern	FACW							Shade		1-3.5'			H	
Pachysandra terminalis Japanese pachysandra	FACU	Mesic	1-2	_	_	_	M	Shade	Evergreen ground cover	_	Fiberous Shallow	No	Low	_
Panicum virgatum switch grass	FAC to FACU	Mesic	2-4	H	_	_	H	Sun or Shade	Grass	4-5'	Fiberous Shallow	Yes	High	Can spread fast and reach height of 6'.
Vinca major large periwinkle	FACU	Mesic	1-2	_	_	_	H	Shade	Evergreen ground cover	_	Fiberous Shallow	No	Low	Sensitive to soil compaction and pH changes.
Vinca minor common periwinkle	FACU	Mesic	1-2	_	_	_	H	Shade	Evergreen ground cover	_	Fiberous Shallow	No	Low	_
Indian grass														
Little bluestem														
Deer tongue														
Green coneflower														

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Appendix C: Recommended Plant Species For Use In Bioretention - Shrub Species

Species	Moisture Regime		Tolerance						Morphology			General Characteristics		Comments
Scientific Name Common Name	Indicator Status	Habitat	Ponding (days)	Salt	Oil/ Grease	Metals	Insects/ Disease	Exposure	Form	Height	Root System	Native	Wildlife	
Aronia arbutifolia (Pyrus arbutifolia) red chokeberry	FACW	Mesic	1-2	H	_	H	M	Sun to partial sun	Deciduous shrub	6-12'	_	Yes	High	Good bank stabilizer. Tolerates drought.
Berberis thunbergii Japanese barberry	FAC	Mesic	2-4	H	H	H	M	Sun	Rounded, broad dense shrub	5-7'	Shallow	No	Med	_
Clethra alnifolia sweet pepperbush	FAC	Mesic to wet Mesic	2-4	H	_	_	H	Sun to partial sun	Ovoid shrub	6-12'	Shallow	Yes	Med	Coastal plain species.
Cornus Stolonifera (Cornus sericea) red osier dogwood	FACW	Mesic-Hydric	2-4	H	H	H	M	Sun or shade	Arching, spreading shrub	8-10'	Shallow	Yes	High	Needs more consistent moisture levels.
Cornus amomum silky dogwood	FAC	Mesic	1-2	L	_	_	M	Sun to partial sun	Broad-leaved	6-12'	_	Yes	High	Good bank stabilizer
Euonymous alatus winged euonymous	FAC	Mesic	1-2	H	H	H	M	Sun or shade	Flat, dense horizontal branching shrub	5-7'	Shallow	No	No	_
Euonymous europaeus spindle-tree	FAC	Mesic	1-2	M	M	M	M	Sun to partial sun	Upright dense oval shrub	10-12'	Shallow	No	No	_
Hamamelis virginiana witch hazel	FAC	Mesic	2-4	M	M	M	M	Sun or shade	Vase-like compact shrub	4-6'	Shallow	Yes	Low	_
Hypericum densiflorum common St. John's wort	FAC	Mesic	2-4	H	M	M	H	Sun	Ovoid shrub	3-6'	Shallow	Yes	Med	_
Ilex glabra inkberry	FACW	Mesic to wet Mesic	2-4	H	H	_	H	Sun to partial sun	Upright dense shrub	6-12'	Shallow	Yes	High	Coastal plain species.
Ilex verticillata winterberry	FACW	Mesic to wet Mesic	2-4	L	M	_	H	Sun to partial sun	Spreading shrub	6-12'	Shallow	Yes	High	_

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Appendix C: Recommended Plant Species For Use In Bioretention - Shrub Species

Species	Moisture Regime		Tolerance						Morphology			General Characteristics		Comments
Scientific Name Common Name	Indicator Status	Habitat	Ponding (days)	Salt	Oil/ Grease	Metals	Insects/ Disease	Exposure	Form	Height	Root System	Native	Wildlife	
Itea virginica tassel-white, Virginia sweetspire	OBL	Mesic	1-2	M	_	_	M	Sun or shade	Broad-leaved, deciduous shrub	6-12'	_	Yes	Low	_
Juniperus communis "compressa" common juniper	FAC	Dry Mesic-Mesic	1-2	M	H	H	M-H	Sun	Mounded shrub	3-6'	Deep taproot	No	High	Evergreen
Juniperus horizontalis "Bar Harbor" creeping juniper	FAC	Dry Mesic-Mesic	1-2	M	H	H	M-H	Sun	Matted shrub	0-3'	Deep taproot	No	High	Evergreen
Lindera benzoin spicebush	FACW	Mesic to wet Mesic	2-4	H	_	_	H	Sun	Upright shrub	6-12'	Deep	Yes	High	_
Myrica pennsylvanica bayberry	FAC	Mesic	2-4	H	M	M	H	Sun to partial sun	Rounded, compact shrub	6-8'	Shallow	Yes	High	Coastal plain species.
Physocarpus opulifolius ninebark	FAC	Dry Mesic to wet Mesic	2-4	M	_	_	H	Sun	Upright shrub	6-12'	Shallow	Yes	Med	May be difficult to locate.
Viburnum cassinoides northern wild raisin	FACW	Mesic	2-4	H	H	H	H	Sun to partial sun	Rounded, compacted shrub	6-8'	Shallow	Yes	High	_
Viburnum dentatum arrow-wood	FAC	Mesic to wet	2-4	H	H	H	H	Sun to partial sun	Upright, multi-stemmed shrub	8-10'	Shallow	Yes	High	_
Viburnum lentago nannyberry	FAC	Mesic	2-4	H	H	H	H	Sun to partial sun	Upright, mlti-stemmed shrub	8-10'	Shallow	Yes	High	_

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Appendix C Recommended Plant Species For Use In Bioretention - Tree Species

Species	Moisture Regime		Tolerance						Morphology			General Characteristics		Comments
Scientific Name Common Name	Indicator Status	Habitat	Ponding (days)	Salt	Oil/ Grease	Metals	Insects/ Disease	Exposure	Form	Height	Root System	Native	Wildlife	
Acer rubrum red maple	FAC	Mesic-Hydric	4-6	H	H	H	H	Partial sun	Single to multi-stem tree	50-70'	Shallow	Yes	High	–
Amelanchier canadensis shadbush	FAC	Mesic	2-4	H	M	–	H	Partial sun	Single to multi-stem tree	35-50'	Shallow	Yes	High	Not recommended for full sun
Betula nigra river birch	FACW	Mesic-Hydric	4-6	–	M	M	H	Partial sun	Single to multi-stem tree	50-75'	Shallow	Yes	High	Not susceptible to bronze birch borer.
Betula populifolia gray birch	FAC	Xeric-Hydric	4-6	H	H	M	H	Partial sun	Single to multi-stem tree	35-50'	Shallow to deep	No	High	Native to New England area.
Fraxinus americana white ash	FAC	Mesic	2-4	M	H	H	H	Sun	Large tree	50-80'	Deep	Yes	Low	–
Fraxinus Pennsylvanica green ash	FACW	Mesic	4-6	M	H	H	H	Partial sun	Large tree	40-65'	Shallow to deep	Yes	Low	–
Ginko biloba Maidenhair tree	FAC	Mesic	2-4	H	H	H	H	Sun	Large tree	50-80'	Shallow to deep	No	Low	Avoid female species-offensive odor from fruit.
Gleditsia triacanthos honeylocust	FAC	Mesic	2-4	H	M	–	M	Sun	Small caopied large tree	50-75'	Shallow to deep variable taproot	Yes	Low	Select thornless variety.
Juniperus virginiana eastern red cedar	FACU	Mesic-Xeric	2-4	H	H	–	H	Sun	Dense single stem tree	50-75'	Taproot	Yes	Very high	Evergreen
Liquidambar styraciflua sweet gum	FAC	Mesic	4-6	H	H	H	M	Sun	Large tree	50-70'	Deep taproot	Yes	High	Edge and perimeter; fruit is a maintenance problem.
Nyssa sylvatica black gum	FACW	Mesic-Hydric	4-6	H	H	H	H	Sun	Large tree	40-70'	Shallow to deep taproot	Yes	High	–

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Appendix C Recommended Plant Species For Use In Bioretention - Tree Species

Species	Moisture Regime		Tolerance						Morphology			General Characteristics		Comments
Scientific Name Common Name	Indicator Status	Habitat	Ponding (days)	Salt	Oil/ Grease	Metals	Insects/ Disease	Exposure	Form	Height	Root System	Native	Wildlife	
Platanus acerifolia London plane-tree	FACW	Mesic	2-4	H	–	–	M	Sun	Large tree	70-80'	Shallow	No	Low	Tree roots can heave sidewalks.
Platanus occidentalis sycamore	FACW	Mesic-Hydric	4-6	M	M	M	M	Sun	Large tree	70-80'	Shallow	Yes	Med	Edge and perimeter; fruit is a maintennance problem; tree is also prone to windthrow.
Populus deltoides eastern cottonnwood	FAC	Xeric-Mesic	4-6	H	H	H	L	Sun	Large tree with spreading branches	75-100'	Shallow	Yes	High	Short lived.
Quercus bicolor Swamp white oak	FACW	Mesic to wet Mesic	4-6	H	–	H	H	Sun to partial sun	Large tree	75-100'	Shallow	Yes	High	One of the faster growing oaks.
Quercus coccinea scarlet oak	FAC	Mesic	1-2	H	M	M	M	Sun	Large tree	50-75'	Shallow to deep	Yes	High	–
Quercus macrocarpa bur oak	FAC	Mesic to wet Mesic	2-4	H	H	H	M	Sun	Large spreading tree	75-100"	Taproot	No	High	Native to Midwest.
Quercus palustris pin oak	FACW	Mesic-Hydric	4-6	H	H	H	M	Sun	Large tree	60-80'	Shallow to deep taproot	Yes	High	–
Quercus phellos willow oak	FACW	Mesic to wet Mesic	4-6	H	–	–	H	Sun	Large tree	55-75'	Shallow	Yes	High	Fast growing oak.
Quercus rubra red oak	FAC	Mesic	2-4	M	H	M	M	Sun to partial sun	Large spreading tree	60-80'	Deep taproot	Yes	High	–
Quercus shumardii Shumard's red oak	FAC	Mesic	2-4	H	H	H	M	Sun to partial sun	Large spreading tree	60-80'	Deep taproot	No	High	Native to Southeast.

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Appendix C Recommended Plant Species For Use In Bioretention - Tree Species

Species	Moisture Regime		Tolerance						Morphology			General Characteristics		Comments
Scientific Name Common Name	Indicator Status	Habitat	Ponding (days)	Salt	Oil/ Grease	Metals	Insects/ Disease	Exposure	Form	Height	Root System	Native	Wildlife	
Robinia pseudo-acacia black locust	FAC	Mesic-Xeric	2-4	H	H	H	M	Sun	Typically tall and slender	30-50'	Shallow	Yes	Low	Edge and perimeter; fruit is a maintenance problem; tree is also prone to windthrow.
Sophora japonica Japanese pagoda tree	FAC	Mesic	1-2	M	M	–	M	Sun	Shade tree	40-70'	Shallow	No	Low	Fruit stains sidewalk.
Taxodium distichum bald cypress	FACW	Mesic-Hydric	4-6	–	–	M	H	Sun to partial sun	Typically single stem tree	75-100'	Shallow	Yes	Low	Not well documented for planting in urban areas.
Thuja occidentalis arborvitae	FACW	Mesic to wet Mesic	2-4	M	M	M	H	Sun to partial sun	Dense single stem tree	50-75'	Shallow	No	Low	Evergreen
Zelkova serrata Japanese zelkova	FACU	Mesic	1-2	M	M	–	H	Sun	Dense shade tree	60-70'	Shallow	No	Low	Branches can split easily in storms.

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GLOSSARY

BIOFILTERS:
Grass depression areas such as engineered channels or swales that are used to collect and filter urban stormwater. This term was developed in the Pacific Northwest.

BIORETENTION:
A water quality practice that utilizes landscaping and soils to treat urban stormwater runoff by collecting it in shallow depressions, before filtering through a fabricated planting soil media.

BEST MANAGEMENT PRACTICE (BMP):
A structural device designed to temporarily store or treat urban stormwater runoff in order to mitigate flooding, reduce pollution and provide other amenities.

COEFFICIENT OF PERMEABILITY:
An engineering constant value which is used to measure the capability of a filter media to pass liquid through a given surface area.

DISCONNECTED IMPERVIOUS SURFACES:
Discontinuous impervious surfaces that allow for the infiltration and filtration of precipitation. An example of this is a residential subdivision in which each dwelling's roof tops drain through a vegetative strip before reaching the road surface.

DRY SWALE:
An open drainage channel or depression, explicitly designed to detain and promote the filtration of stormwater runoff into an underlying soil media.

EDGE EFFECT:
Extensive, well-defined edges between the impervious and pervious surfaces.

EXFILTRATE:
The downward movement of runoff through the bottom of a treatment system into the soil layer.

FIBRIC PEAT:
Organic material, usually derived from wetland vegetation, in which the undecomposed fibrous organic materials are easily identifiable. Also characterized by low bulk density and a highly porous structure.

FILTER BED CHAMBER:
The section of a constructed filtration device that houses the filter material and the outflow piping.

FILTER STRIPS:
A vegetated area that treats sheetflow and/or interflow by removing sediment and other pollutants. The area may be grass-covered, forested or of mixed vegetative cover (e.g. wildflower meadow).

FOREBAY:
Additional storage space located near a stormwater BMP inlet that serves to trap incoming coarse sediments before they accumulate in the main treatment area.

FREEBOARD:
The space from the top of an embankment or a channel bank to the highest water elevation expected for the largest storm designed to be stored or conveyed. The space is required as a

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safety margin in a pond, basin or channel.

GEOTEXTILE FABRIC:
A synthetic textile of relatively small mesh or pore size that is used to (a) allow water to pass through while keeping sediment out (permeable), or (b) prevent both runoff and sediment from passing through (impermeable). Also known as filter fabric.

HEAD:
Height or water elevation above a given location and the pressure exerted by it due to gravity.

HEMIC PEAT:
An organic material, usually derived from wetland vegetation that is moderately decomposed, has a moderate bulk density and modest porosity.

HERBICIDES:
Chemicals developed to control or eradicate plants.

HIGH-INPUT LAWN:
A heavily irrigated lawn subject to high usage of chemicals: fertilizers, pesticides and fungicides.

HUMIC:
A soil or other material characterized by a high organic content.

IMPERVIOUS:
The characteristic of a material which prevents the infiltration or passage of liquid through it. This may apply to roads, streets, parking lots, rooftops and sidewalks.

INFLOW REGULATION:
The control, usually by an engineering device, of the inflow into a BMP.

INSECTICIDES:
Chemicals developed to control or eradicate insects.

LOW-INPUT LAWN:
A lawn that is regularly mowed but is not subjected to a high usage of chemicals and irrigation.

MICRO-ENVIRONMENT:
This term refers to the conditions created under which a separate, smaller environment exists distinct from the dominant one, which can affect and be affected by the immediate surroundings.

NPDES:
Acronym for the National Pollutant Discharge Elimination System, which regulates point source and stormwater discharge. NPDES standards were promulgated by the EPA in accordance with the Clean Water Act.

OFF-LINE:
A water quality system designed by diverting stormwater from a stream or storm drainage system.

ON-LINE:
A water quality system designed to retain stormwater in its original stream channel or storm drainage system.

OPEN VEGETATED CHANNELS:
Also known as swales, grass channels, and biofilters. These systems are used for the conveyance, retention, infiltration and filtration of stormwater runoff.

PEA GRAVEL CURTAIN DRAIN:
A thin wall of small, river-run gravel used to convey water to the sides/bottom of bioretention practices.

PEA GRAVEL DIAPHRAGM:
A stone trench filled with small, river-run gravel used as pretreatment and inflow regulation in stormwater filtering systems.

PEAK DISCHARGE (FLOW RATE):
The maximum instantaneous rate of flow during a storm, usually in reference to a specific design storm event.

PERMISSIBLE VELOCITY:
The maximum rate of flow allowable for vegetated open channels, before erosive channel conditions occur.

PERVIOUS:
Any material that allows for the passage of liquid through it.

PUBLIC TURF:
Pervious land held in the public domain. Examples include parks, golf courses, cemeteries, median strips and school grounds.

RUNOFF PRETREATMENT:
Technique employed in a stormwater BMP to retain storage volumes or prevent clogging by trapping coarse materials before they enter the system.

RAINFALL FREQUENCY SPECTRUM:
The frequency distribution of cumulative rainfall volume generated by all storm events. This analysis is used to determine how much rain can be treated in a stormwater filter, and how much may be bypassed.

RATE-BASED DESIGN:
BMP design which uses the discharge in volume per unit of time as a basis for sizing the practice.

RESIDENTIAL SUBDIVISION:
A large land area divided into smaller parcels for the purpose of housing.

RETROFIT:
The installation of a new BMP or the improvement of an existing one in a previously developed area.

RUNON:
The flow of stormwater from impervious cover to pervious cover.

SAND FILTER:
A stormwater quality treatment practice, whereby runoff is diverted into a self-contained bed of sand. The runoff is then strained through the sand, collected in underground pipes and returned back to the stream or channel.

SARA:
An acronym for Congressional legislation referring to the Superfund Amendments and Reauthorization Act of 1980.

SEDIMENTATION CHAMBER:
This is a section of a BMP that provides for the settling out of large particles from suspension.

SHEET FLOW:
Stormwater flowing sheet-like over pervious or impervious surfaces.

SHORT-CIRCUITING:
The passage of runoff through a BMP in a timespan or flowpath without adequate treatment.

STORMWATER HOT SPOTS:
Land-uses or activities that generate highly contaminated runoff. Examples include fueling stations and airport de-icing facilities.

STORMWATER INFILTRATION SYSTEMS:
BMP's which are designed to percolate runoff into the underlying soil.

STORMWATER PONDS:
A land depression created for the detention or retention of stormwater runoff.

STORMWATER WETLAND:
A shallow, constructed pool that captures stormwater and allows for the growth of characteristic wetland vegetation.

SUBMERGED GRAVEL FILTERS:
A filtering BMP which uses a gravel based substrate, supporting a wetland vegetation cover crop, to treat urban runoff.

TIME OF CONCENTRATION:
An engineering term representing the travel time of runoff through a watershed, subwatershed or catchment.

ULTRA-URBAN:
A region dominated by highly developed areas in which very little pervious surfaces exist.

UNDERDRAIN SYSTEM:
A perforated pipe system in a gravel bed, installed on the bottom of filtering BMP's, which are used to collect and remove filtered runoff.

VOLUME-BASED DESIGN:
A BMP design which uses the volume of runoff as a basis for sizing the practice.

VOLUMETRIC RUNOFF COEFFICIENT (RV):
A value that is applied to a given rainfall volume to yield a corresponding runoff volume.

WET SWALE:
An open drainage channel or depression, explicitly designed to retain water or intercept groundwater for water quality treatment.

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SAND FILTER FOR TREATING STORM WATER RUNOFF

Revision 5/99
Process Code: Navy and Marine Corps: SR-16-99; Air Force: FA08; Army: N/A
Usage List: Navy: Low; Marine Corps: Low; Army: Low; Air Force: Low
Alternative for: Wet ponds, Infiltration basins, Infiltration trenches, extended detention dry ponds, vegetated filter strips, grass swales
Compliance Areas: Low
Applicable EPCRA Targeted Constituents: Lead (CAS: 7439-92-1); Phosphorus (CAS: 7723-14-0)

Overview: Sand filters can be used for storm water quality control and managing storm water runoff volumes. Sand filters are composed of at least two components: a sedimentation chamber and a filtration chamber. The sedimentation chamber removes floatables and heavy sediments, while the filtration chamber removes additional pollutants by filtering flow through a sand bed. Treated filtrate is normally diverted back to the storm drainage system via an underdrain system or pipe network. Pollutants such as suspended solids, biochemical oxygen demand (BOD), total phosphorus, and fecal coliform bacteria are effectively removed from storm water flows when treated by a sand filter system. Other pollutants removed include phosphorus and metals. Sand filter designs include the surface sand filter basin (AKA Austin sand filter), the underground vault sand filter (Washington, DC sand filter), the double trench sand filter (Delaware sand filter), the stone reservoir trench sand filter, and the peat sand filter system. Modifications are often made to these designs based on site-specific conditions.

Sand filters provide a highly effective means of removing pollutants from storm water while remaining flexible in application to allow for modifications in basic design structure to accommodate site-specific criteria. Modifications to the basic structure arise due to site differences, including drainage area served, filter surface areas, land requirements, and quantity of runoff treated. Sand filters are currently popular best management practices (BMPs) used in Delaware; Florida; Austin, Texas; Alexandria, Virginia; and Washington, DC.

The Austin sand filter was designed to detain runoff in a sedimentation chamber where heavy sediments and floatables are removed. Estimates of pollutant removal efficiencies for various Austin sand filters, based on the preliminary findings of the city's storm water monitoring program, are as follows. In addition, data from an intermittent sand filter (Delaware sand filter), constructed to treat runoff from a commercial parking lot near National Airport in Alexandria, Virginia, are provided below.

Pollutant	Austin sand filter	Delaware sand filter
Fecal Coliform	76 %	not measured
Total Suspended Solids (TSS)	70 %	80-83%
Biochemical Oxygen Demand (BOD)	70 %	77.5%
Total Organic Carbon (TOC)	48 %	65.9%
Total Kjeldahl Nitrogen (TKN)	46 %	70.6%
Iron (Fe)	45%	not measured
Lead (Pb)	45 %	not measured
Zinc (Zn)	45 %	81.6%
Total Phosphorous (TP)	33 %	72.3%
Total Nitrogen	21 %	47.2%
Nitrate as Nitrogen (NO_3--N)	0 %	62.7%

The percentages listed for the Austin sand filter include partial and full sedimentation systems with different drainage areas. Current monitoring data from the Austin sand filters indicates phosphorous removal efficiencies of up to 60 percent. The Austin sand filter also has been used in Alexandria, Virginia; monitoring of these units indicated a phosphorus removal of up to 40 percent. Nitrate was not removed nor is it known what the removal efficiencies are for other dissolved pollutants.

Performance of sand filters may be sustained through frequent inspections and replacement of the filter fabric and the top of the media every three to five years, depending on the pollutant load being treated. One system has been reported to need filter changes two times per year due to heavy pollutant loads. Accumulated trash and debris should be removed from the sand filters every six months or as necessary. Performance also can be increased by stabilizing the drainage area to minimize sediment loading, ensuring that the sedimentation chamber adequately removes suspended solids and sediments prior to the filtration chamber and allowing for adequate detention times for both sedimentation and filtration.

The design of sand filters with impermeable chambers that prevent groundwater infiltration are preferred in situations where groundwater contamination is a concern. The Austin; Delaware; and Washington, DC, sand filters may substitute for water quality inlets when hydrocarbons are of concern. Due to the size of the Austin sand filter, it also can be used instead of wet ponds for treatment of contaminated run-off in areas where evaporation exceeds rainfall.

Compliance Benefit:

The use of a sand filter for treating stormwater runoff may help facilities meet requirements for implementation of stormwater runoff best management practices contained in stormwater permits and plans (**40 CFR 122.26**).

The compliance benefits listed here are only meant to be used as a general guideline and are not meant to be strictly interpreted. Actual compliance benefits will vary depending on the factors involved, e.g. the amount of workload involved.

Materials Compatibility:

No materials compatibility issues were identified.

Safety and Health:

Safety and health concerns are dependent on the types of contaminants in the stormwater. Metals and phosphorus, for instance, require caution in handling. They are skin irritants. Protective gear should be worn when handling contaminants such as fecal coliform. Proper personal protection equipment is, therefore, recommended. In addition, care should be taken when working with sealed systems, as gas may accumulate.

Consult your local industrial health specialist, your local health and safety personnel, and the appropriate MSDS prior to implementing any of these technologies.

Benefits:

- Sand filters, in particular those mentioned previously, achieve high removal efficiencies for suspended solids, BOD, and fecal coliform bacteria, and total phosphorous.
- Hydrocarbons and nutrients also are removed by sand filters.
- Sand filters designed with impermeable basins limit the potential for groundwater contamination while treating storm water.
- Sand filters can be used in small sites (*e.g.,* gas stations or other urban settings) where a wet pond is not possible due to spacial constraints.

Disadvantages:

- Nitrates are not removed.
- Sand filters are ineffective in removing dissolved pollutants except by adsorption.

Economic Analysis:

Construction costs vary depending on the sand filter system being designed. The Austin sand filtration system costs approximately $18,500 for treatment of a 1-acre drainage area. In this instance, the cost decreases with increasing drainage area. Cost per acre decreases as the number of acres served

increases. For example, the cost for a sand filter decreases to approximately $2,360/acre when treating 30 acres. The precast cost for one impervious acre for a Washington, DC, sand filter is approximately $25,000 to $30,000. Costs for the Delaware sand filter are $20,000 per impervious acre treated.

Assumptions:

- The following comparison uses an average cost of sand filters.
- Maintenance costs for a sand filter average 5 percent of the construction cost.
- Labor costs for operating a wet pond include mowing and debris removal.
- Labor costs for operating a sand filter include filter changing, gravel and sand replacement, and debris removal, estimated at approximately 3 hours/year.
- Material costs for the gravel layer, filter fabric, and top portion of sand based on the experience of the Washington, D.C., sand filter are approximately $1,700 annually.
- The figures in the table are based on one impervious acre treated.
- The cost per pound of pollutant removed equals $8.30 based on the experience for the Washington, D.C. sand filter.
- Labor costs $45/hour.
- Materials disposed are not hazardous. Solid waste disposal costs are $40/ton or $0.02/lb.
- Annual disposal of sediments is 350 lbs.

Annual Operating Cost Comparison for
Wet Ponds and Sand Filter

	Wet Pond	Sand Filter
Operational Costs:		
Labor:	$1,620	$135
Materials:	$0	$1,700
Waste Disposal	$7	$7
Total Costs:	$1,627	$1,842
Total Income:		
Annual Benefit:	-$1,627	-$1,842

R0022187

Economic Analysis Summary

Annual Savings:	$-215
Capital Cost for Equipment/Process:	$20,000
Payback Period for Investment in Equipment/Process:	N/A

Overall costs for installing and operating a sand filter system appear to be higher than that of using a wet pond system. However, in many urban situations, it is not feasible to install a wet pond. A sand filter is an effective alternative for treating stormwater runoff.

Click Here to View an Active Spreadsheet for this Economic Analysis and Enter Your Own Values. To return from the Active Spreadsheet, click the ***reverse arrow*** in the Tool Bar.

NSN/MSDS:

Product	NSN	Unit Size	Cost
None Identified			

Approval Authority: Approval is controlled locally and should be implemented only after engineering approval has been granted. Major claimant approval is not required.

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R0022188

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Vendors: N/A

Sources:

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R0022189

VORTEX SOLIDS SEPARATORS FOR TREATING STORM WATER RUNOFF

Revision 5/99
Process Code: Navy: N/A; Marine Corps: N/A; Air Force: N/A; Army: N/A
Usage List: Navy: Low; Marine Corps: Low, Army: Low; Air Force: Low
Alternative for: N/A
Compliance Areas: Low
Applicable EPCRA Targeted Constituents: N/A

Overview: A vortex solids separator is a wet-weather treatment unit, containing no moving parts, designed to remove solids and floatables from wastewater or storm water physically. The unit is cylindrical in design so that, as flow enters the unit tangentially, it induces a swirling vortex which concentrates solids at the bottom of the unit in the underflow. Vortex solids separators can be used for both combined sewer overflows (CSOs) and separate storm sewers. During CSO events, concentrated solids are collected and removed from the bottom of the unit through a sanitary sewer and discharged to a wastewater treatment plant. During separate storm water events, concentrated solids can be removed through the bottom of the unit and sent to a holding tank or pond, where further sedimentation will take place. Clarified effluent exits through the top of the unit and is returned to the receiving water. Vortex units can be installed on line or off line in a system. The units can also be designed to be used individually or as group of units (e.g. in parallel at high flows and series at lower flows). Vortex units can be designed to be used in combination with other best management practices (BMPs).

The design of a vortex solids separator should be based on the anticipated type and quantity of pollutants to be removed, as well as the settleability characteristics of those pollutants. The quantity of flow to be treated should be established prior to the design phase to achieve the desired treatment level. Pilot-scale testing should be conducted during the design phase to determine the swirl treatability at each site.

Unit performance is based on the vortex separation mechanism for which each type has its own design criteria for solids/liquids separation. The design criteria for the Swirl, which is available to the public from the EPA, is based on settleability studies developed in the 1970s. Design specifications and pilot-scale treatability studies are required for each site-specific application.

Data collected from solids removal studies indicate vortex solids separators are effective at removing gritty materials, heavy particulates, and floatables from

waste water flow, but ineffective in removing materials with poor settleabilities. The net solids have been calculated for various units in use for CSO applications. Net solids accounts for the large underflow volume and solids collected as a result of swirl concentrating. Net solids removal ranged between 7 and 34 percent. The following table presents average performance characteristics of commercial Vortex Separators collected from four units.

Unit	Location	Effluent Hydraulic Loading Rate (gpm/sf)	Volume Reduction %	Total Solids Removal %	Net Solids Removal %
Swirl	Washington, DC	10	24	38	12
Fluid sep	Tengen, Germany	11	47	54	7
Storm King	James Bridge, UK	7.5	39	53	14

Disadvantages of vortex solids separators are limited effectiveness as evidenced by comparing percent solids to net solids removal. Vortex separators have an underflow that requires further treatment. The documented removal rates for vortex separators may not meet water quality treatment objectives for proposed locations. Little information is available for vortex solids separators treating pollutants other than solids.

An advantage of vortex separators is the small land requirement as compared to other BMPs (i.e., wet ponds). Vortex unit construction constraints include the ability of the soil to support the unit, the depth of soil, and the slope of the site, which may determine whether an above- or below-ground unit is used. Maintenance for most separators includes a wash down after every CSO event to prevent odors. This would not be necessary for a storm water application. Some units have a self-washing mechanism.

Compliance Benefit:

The use of a vortex solids separator for treating storm water runoff may help facilities meet requirements for implementation of storm water runoff best management practices contained in stormwater permits and plans (**40 CFR 122.26**). In addition, the vortex solids separators can be used in combined sewer overflows to help facilities meet pretreatment standards for discharges of wastewater to a POTW (**40 CFR 403**).

The compliance benefits listed here are only meant to be used as a general guideline and are not meant to be strictly interpreted. Actual compliance benefits will vary depending on the factors involved, e.g. the amount of workload involved.

Materials Compatibility: N/A

Safety and Health: Proper design, operation, and maintenance of the equipment is required for its safe use.

Consult your local Industrial Health specialist, your local health and safety personnel, and the appropriate MSDS prior to implementing any of these technologies.

Benefits:

- Vortex solids separators are able to separate solids and floatables from storm water and wastewater using a swirling vortex. They can be used in instances where a separation technology is limited by space or land constraints, such as surface slope or soil composition. Vortex separators have no moving parts and, therefore, are not maintenance intensive.

Disadvantages:

- Vortex solids separators have limited effectiveness in use with wet-weather flows
- May not meet water quality treatment objectives for some locations
- Limited information is available for vortex solids separators treating pollutants other than solids

Economic Analysis: Budgeting for construction of a vortex separator unit should include predesign costs, capital costs, and operation and maintenance costs. As of 1997, the predesign costs for a Storm King are typically $21,000, and between $27,000 and $106,000 for the Fluidsep. Settleability curves published for the Swirl can be used as the basis for design and eliminate predesign costs. Capital costs for vortex solid separator treatment facilities in the US are site specific and vary between $3,200 and $5,600 per acre of drainage basin. The capitol cost for an individual unit alone is approximately $5,200 per mgd.

Energy requirements for most vortex solid separators are nil unless the facility requires pumping. Washdown costs for vortex separators primarily include labor or energy costs for an automatic washdown. The Surrey Heath Storm King facility lacks a foul sewer line and collects residuals in a collection zone.

These residuals are periodically emptied every 2 to 3 years, at an estimated cost of between $300 and $500 per cleaning.

NSN/MSDS:

Product	NSN	Unit Size	Cost
None Identified			

Approval Authority: Navy: Approval is controlled locally and should be implemented only after engineering approval has been granted. Major claimant approval is not required.

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Vendors: The following is a list of vendors. This is not meant to be a complete list, as there may be other manufacturers of this type of equipment.

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P.O. Box 366
Scarborough, ME 04070-0366
Phone: (800) 848-2706
Contact: Mr. Steve Hides

Hansj rg Brombach
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Phone: (07931)97 10-0

Sources:

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R0022194

JOINT SERVICE POLLUTION PREVENTION OPPORTUNITY DATA SHEET

WATER QUALITY INLETS TO CONTROL STORM WATER RUNOFF

Revision 5/99
Process Code: Navy and Marine Corps: SR-16-99; Air Force: FA08; Army: N/A
Usage List: Navy: Low; Marine Corps: Low; Army: Low; Air Force: Low
Alternative for: Direct Stormwater Discharge
Compliance Areas: Low
Applicable EPCRA Targeted Constituents: N/A

Overview:

Water quality inlets (WQIs) separate pollutants from the first flush of storm water using a series of chambers for pollutant sedimentation, screening, and separation. A water quality inlet typically consists of a sedimentation chamber, an oil separation chamber, and a discharge chamber. Because of their separation capabilities, WQIs are occasionally referred to as oil/grit separators or oil/water separators. These best management practices (BMPs) may be constructed on site, precast, or manufactured by a vendor.

Water quality inlets are widely used in the US for improving storm water runoff quality where space is limited and funding prohibits the use of larger BMPs, such as ponds or wetlands. Water quality inlets effectively separate hydrocarbons from storm water runoff. During the first 5 years of use, over 95% of all WQIs were in operation as designed. WQIs do not manage the volume of storm water flow, due to limited capacity, and have limited removal efficiencies when not properly maintained. For these reasons, WQIs are often used to pretreat runoff prior to discharge to other BMPs.

The WQI should be located within close proximity to a storm drain network to allow for future discharge from the WQI to the sewer system. WQIs are typically used as an off-line treatment process where lower flows will be encountered; high flows result in resuspension of settled material. Construction activity should be complete and the drainage area stabilized to minimize sediment loading to a WQI. The WQI should be located in a small, impervious area and be watertight.

Storm water runoff enters the sedimentation chamber in a water quality inlet where coarse materials settle. Flow from the sedimentation chamber is conveyed to the second chamber through an orifice covered with a trash rack and located halfway down the wall separating the two chambers. The second chamber functions as an oil separation chamber. Water that enters the third sequential chamber discharges through a storm water outlet pipe. The design should include permanent pools within the chambers, to reduce sediment resuspension during storm events, and manholes above the chambers, to provide access for cleaning and inspection.

The limited capacity of most WQIs typically means that the discharge rate is high and the detention time is relatively short. Most water quality inlets have an average detention time of less than a half-hour. Efficient pollutant removal is dependent upon proper maintenance; the lack of proper maintenance may result in resuspension and discharge of settled pollutants and separated oil. The required maintenance will vary from site to site, but cleaning before the start of each season and inspection after every storm event should ensure proper functioning of the WQI.

Water quality inlets generally have minimal effect on the removal of nutrients, metals, and organic pollutants other than free petroleum products. The sedimentation chamber can be expected to reduce grit and sediments partially. Separation of dissolved or emulsified oil from water is rarely achieved, although WQIs are effective in separating free oil and grease from storm water.

Compliance Benefit:

The use of water quality inlets for treating storm water runoff may help facilities meet the requirements for implementation of storm water runoff Best Management Practices (BMPs) contained in stormwater permits and plans (**40 CFR 122.26**).

The compliance benefits listed here are only meant to be used as a general guideline and are not meant to be strictly interpreted. Actual compliance benefits will vary depending on the factors involved, e.g. the amount of workload involved.

Materials Compatibility:

No material incompatibilities identified.

Safety and Health:

The safety and health issues depend on the types of contaminants in the storm water. Some hydrocarbons may be a minor irritant to mucous membranes and eyes. Handling hydrocarbons requires caution; proper personal protective equipment is recommended.

Consult your local Industrial Health Specialist, your local health and safety personnel, and the appropriate MSDS prior to implementing any of these technologies.

Benefits:

- WQIs are useful for separating sediments and oil from storm water runoff. If properly maintained, these pollutants are removed in the WQI, and the quality of the downstream storm water is improved.

Disadvantages:

- WQIs do not manage large volumes of storm water
- Limited removal efficiencies when not properly maintained
- WQI residuals may require disposal as a hazardous waste.

Economic Analysis:

The cost for a precast WQI ($5,000 - $35,000) is generally lower than for other units. The cost varies depending on the capacity of the inlet and the complexity of the system. A cast-in-place WQI also varies in cost. The cost of a large oil/water separator that services the air field storm drains at Westover Air Reserve Base was estimated to cost approximately $380,000 for labor and materials.

Assumptions:

- Labor Rate: $45/hour.
- Cost to develop procedures (estimated at 4 man-hours) for maintenance, and for disposal.
- Inspect separator four times a year (0.5 hours/each).
- Twice a year remove oil/water from WQI.
- Remove 200 gallons of oil/water a year from WQI.
- Oil/grease analysis done at each cleaning of WQI, $40/each.
- Cost of disposal is $0.42/gallon to dispose of oil/water plus an additional $80 for a vacuum truck to come to the facility.
- Once a year remove sediment from WQI.
- Annual disposal of sediments is 350 lbs at $0.02/lb.
- Requires two people four hours to clean separator when removing sediment.
- A Labor cost for operating a sand filter includes filter changing, gravel and sand replacement, and debris removal, which takes approximately 3 hours/year.
- Material costs for the gravel layer, filter fabric, and top portion of sand for sand filter are approximately $1,700 annually.
- The figures in the table are based on one impervious acre treated.

R0022197

Cost Comparison for
Water Quality Inlets vs. Sand Filter

	WQI	Sand Filter
Capital and Installation Costs	$20,000	$20,000
Operational Costs:		
Labor Costs	$630	$135
Materials	$0	$1,700
Waste Disposal Costs	$251	7
Permitting and Lab Analysis Costs	$80	$0
Total Costs (not including capital and installation costs)	$961	$1,842
Total Income:	$0	$0
Annual Benefit:	-$961	-$1,842

Economic Analysis Summary

Annual Savings for WQI:	$881
Capital Cost for Diversion Equipment/Process:	$20,000
Payback Period for Investment in Equipment/Process:	>20 years

Click Here to View an Active Spreadsheet for this Economic Analysis and Enter Your Own Values. To return from the Active Spreadsheet, click the ***reverse arrow*** in the Tool Bar.

NSN/MSDS:

Product	**NSN**	**Unit Size**	**Cost**
None Identified			

Approval Authority: Approval is controlled locally and should be implemented only after engineering approval has been granted. Major claimant approval is not required.

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Air Force:
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Westover Air Reserve Base
250 Patriot Avenue, Suite 1
Chicopee, MA 01022-1638
Phone: (413) 557-2484, DSN 589-2484

Vendors: The following is a list of oil/water separator vendors. This is not intended to be a complete list, as there are numerous manufacturers of this type of equipment.

Jay R. Smith Mfg. Co. (Ultracept)
Environmental Products Group
2781 Gunter Park Dr.
P.O. Box 3237
Montgomery, AL 36109
Phone: (800) 767-0466

XERXES Corp.
7901 Xerxes Avenue South
Minneapolis, MN 55431-1253
Phone: (612) 887-1890

National Fluid Separators, Inc.
827 Hanley Industrial Court
St. Louis, MO 63144
Phone: (314) 968-2838
URL: http://www.mjind.com

Sources:

Mr. Matthew Jabloner, Engineering Field Activity, Northwest, January 1999.
Ms. Gina Rosse-Linderme, Westover Air Reserve Base, January 1999.
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Metropolitan Council of Governments (MWCOG), 1993. The Quality of Trapped Sediments and Pool Water within Oil Grit Separators in Suburban Maryland. Interim Report.
Mr. Jim McPhee, United Industrial Services, Wyoming, RI, July 1997.
Joint Services Pollution Prevention Technical Library Fact Sheet on Sand Filter For Treating Storm Water Runoff, June 1997.

R0022199

WET DETENTION PONDS TO TREAT STORM WATER RUNOFF

Revision: 5/99
Process Code: Navy and Marine Corps: SR-16-99; Air Force: FA08; Army: N/A
Usage List: Navy: Low; Marine Corps: Low; Army: Low; Air Force: Low
Alternative for: Direct Storm Water Discharge
Compliance Areas: Low
Applicable EPCRA Targeted Constituents: Heavy Metals; Phosphorus (CAS: 7723-14-0)

Overview: A wet detention pond is a constructed storm water retention pool wherein physical, biological, and chemical processes remove pollutants from storm water runoff. Pollutants removed include suspended solids, organic matter, dissolved metals, and nutrients. In addition, wet detention ponds control storm water flow which prevents downstream flooding. As storm water enters the pond, treated water is displaced and discharged into a receiving body of water. Enhanced treatment of storm water runoff can be achieved through extended detention and the use of aquatic plants on the perimeter of the pond. Sediment removal can also be increased through the use of a sediment forebay.

Before construction of a pond begins, local, state, and federal permits should be in place for all aspects of construction including wetlands, water quality, dam safety, grading, erosion control, and land use. Wet detention ponds rely on the maintenance of a permanent pool of water within the pond and therefore, should be placed in areas with adequate baseflow from groundwater or from the drainage area to maintain the permanent pool. Soils under the pond should have a low permeability (10^{-5} to 10^{-6} cm/sec) to maintain a permanent wet pool. Pond placement should optimize reuse of a topographic area that allows for maximum detention while requiring minimal earth removal, thus lowering construction costs. Pond construction should not be undertaken near utilities or underlying bedrock.

Pollutant removal in the pond is achieved through one of two methods: solids settling and eutrophication. The solids settling method relies on pollutant removal through sedimentation. The eutrophication method removes nutrients using natural biological processes. According to the National Urban Runoff Program (NURP), up to 2/3 of the suspended sediments, trace metals, and nutrients settle out within 24 hours. Other studies, examining biological removal, suggest hydraulic residence times (HRTs) of close to 2 weeks are required for phosphorus removal.

Documented removal efficiencies for wet detention ponds are as follows:

	Percent Removal	
Parameter	Schueler, 1992[1]	Hartigan, 1988[2]
Total Suspended Solids	50-90	80-90
Total Phosphorous	30-90	
Soluble Nutrients	40-80	50-70
Lead		70-80
Zinc		40-50
Biochemical Oxygen Demand		20-40
Chemical Oxygen Demand		20-40

[1] hydraulic residence time varies
[2] hydraulic residence time of 2 weeks

Two ratios are useful for predicting pollutant removal efficiencies: volume ratio and area ratio. Volume ratio (VB/VR) is the ratio of permanent pool storage (VB) to the mean storm runoff (VR). Area ratio (A/As) is the ratio of the contributing drainage area (A) to the permanent pool surface area (As). Both of these ratios are correlated with treatment efficiencies. Large volume ratios result in increased retention and treatment between storms while low pollutant efficiencies are achieved with low volume ratios.

Pool depth can play a critical role in pollutant removal and storage, but caution should be taken when increasing the depth of the pool. A pond with an HRT of 2 weeks would function optimally at depth ranges from 3 to 8 feet; shallower depths with the same pond surface area have shorter HRTs.

Water within the pond is discharged through a wet pond outlet. A wet pond outlet consists of a vertical riser, either concrete or corrugated metal, attached to a horizontal barrel that conveys storm water flow under the embankment to a receiving stream. The outlet is designed to pass excess water while maintaining a permanent pool. Risers are typically placed in or on the edge of the embankment and are capped with a trash rack to prevent clogging.

As with any storm water best management practice (BMP), proper maintenance will ensure continued proper functioning of the wet detention pond. Proper maintenance may include any or all of the following:

- Clearing trash and debris
- Conducting routine inspections of the embankment and spillway to check structural integrity and look for signs of erosion or animal habitation
- Conducting periodic repairs on the embankment, emergency spillway, inlet, and outlet
- Removing sediment and algae

- Removing woody vegetation or trees from the embankment that could potentially weaken the embankment
- Maintaining the outfall area (i.e., replacing rip-rap, removing sediments, etc.)

Sediments collected by the wet detention pond typically meet toxicity limits and can be landfilled safely. Testing of the sediments may be required if the upstream drainage area is industrial and/or results in highly contaminated runoff. Non-toxic sediments can also be disposed on site, but away from the shoreline to prevent their re-entry into the pond. The removal of sediments in a pond may be necessary every 20 years. This may be decreased to every 50 years if a sediment forebay is used prior to the wet pond. The sediment forebay would require maintenance every 5 to 7 years or when 50% of forebay capacity is silted.

Compliance Benefit:

The use of wet detention ponds for treating storm water runoff may help facilities meet requirements for implementation of storm water runoff best management practices contained in stormwater permits and plans (**40 CFR 122.26**).

The compliance benefits listed here are only meant to be used as a general guideline and are not meant to be strictly interpreted. Actual compliance benefits will vary depending on the factors involved, e.g. the amount of workload involved.

Materials Compatibility:

No materials incompatibilities identified.

Safety and Health:

Proper design, operation, and maintenance of the equipment is required for its safe use. Consult your local Industrial Health specialist, your local health and safety personnel, and the appropriate MSDS prior to implementing any of these technologies.

Benefits:

- Wet detention ponds can decrease the potential for downstream flooding and streambank erosion, and provide improved downstream water quality.
- Water quality is improved through removal of suspended solids, metals, and dissolved nutrients using natural biological and physical processes.
- Properly designed and maintained wet detention ponds can also enhance landscape aesthetics as well as provide wildlife habitat.

Disadvantages:

- Pond construction should not be conducted near utilities or underlying bedrock.
- Sediments from upstream industrial or highly contaminated runoff areas may constitute a hazardous waste requiring special disposal/treatment.
- In Maryland, use of wet ponds is strongly discouraged in cold water fisheries (streams) due to potential thermal impacts.

Economic Analysis:

Budgeting for construction of a wet detention pond should include costs for permitting, designing, constructing, and maintaining the pond. Costs will vary for permitting from state to state, as will requirements concerning pond construction in a developing area versus a developed one. Developing areas tend to be less costly, as there are fewer problems presented by existing utility and other constraints. A publication review indicated an average cost for a 1acre, 5 foot deep pond with a storage volume of 180,000 cubic feet is $75,000.

Fort Meade installed five storm water wet detention ponds in 1997. The cost of the ponds ranged from $4 -7 million. The average pond size is approximately two acres with a depth of one foot.

Literature indicates that annual maintenance and operational costs typically range between 3 to 5 percent of construction costs. Maintenance costs include sediment removal, grass mowing, nuisance control (problematic animals), trash removal, and routine inspections. On-site sediment disposal should be utilized when possible, as costs can be reduced by as much as 50 percent. The operation and maintenance costs at Fort Meade are estimated to be less than 1 percent of the total construction costs.

Assumptions:

- Labor rate: $45/hour.
- Labor costs for operating a wet pond include mowing and debris removal requiring an estimated three hours per month
- Labor costs for operating a sand filter includes filter changing, gravel and sand replacement, and debris removal, requiring an estimated 3 hours/year.
- Material costs for the gravel layer, filter fabric, and top portion of sand for sand filter are approximately $1,700 annually.
- The figures in the table are based on one impervious acre treated.
- Annual disposal of sediments is estimated at 350 lbs. at $0.02/lb.

Cost Comparison for
Wet Detention Ponds vs. Sand Filters

	Wet Detention Pond	Sand Filter
Capital and Installation Costs	$18,000	$20,000
Operational Costs:		
Labor	$1,620	$135
Materials	$0	$1,700
Sediment Disposal	$7	$7
Total Costs (not including capital and installation costs)	$1,207	$1,842
Total Income:	$0	$0
Annual Benefit:	- $1,627	-$1,842

Economic Analysis Summary

Annual Savings for Wet Detention Pond:	$215
Capital Cost for Diversion Equipment/Process:	$18,000
Payback Period for Investment in Equipment/Process:	>30 years

Based on the above analysis it appears that the operational cost of a sand filter system is similar to that of a wet pond system. However, a wet pond system has a high land requirement. Where land is available, wet pond systems may be more appropriate.

Click Here to View an Active Spreadsheet for this Economic Analysis and Enter Your Own Values. To return from the Active Spreadsheet, click the ***reverse arrow*** in the Tool Bar.

NSN/MSDS:

Product	**NSN**	**Unit Size**	**Cost**
None Identified			

Approval Authority: Approval is controlled locally and should be implemented only after engineering approval has been granted. Major claimant approval is not required.

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Vendors: Contact local construction or A&E firms for more information

Sources:

Mr. Bill Harmeyer, Fort Meade, Maryland, January 1999.
Hartigan, J. P. 1988. "Basis for Design of Wet Detention Basin BMPs" in Design of Urban Runoff Quality Control. American Society of Engineers.
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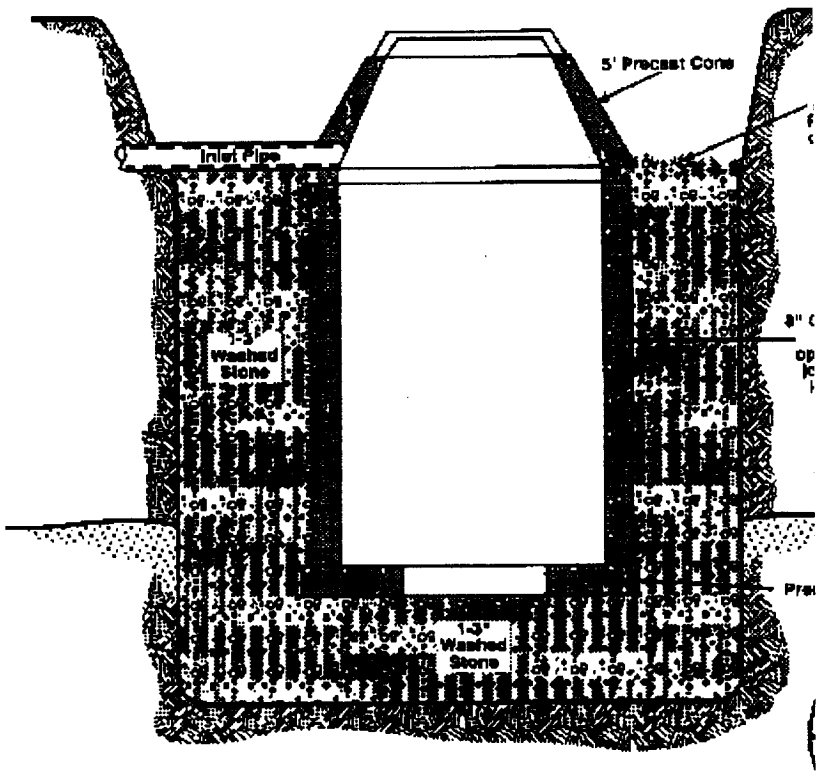
Southeastern Wisconsin Regional Planning Commission (SEWPRC), 1991. Costs for Urban Nonpoint Source Water Pollution Control Measures. Technical Report No. 31.
Urbonas, Ben and Peter Stahre, 1993. Stormwater Best Management Practices and Detention for Water Quality, Drainage, and CSO Management. PTR Prentice Hall, Englewood Cliffs, New Jersey.
Joint Services Pollution Prevention Technical Library Fact Sheet, Sand Filter for Treating Storm Water Runoff, June 1997.
Maryland Department of the Environment, http://www.mde.state.md.us

Office of Water (4601)

Guidance on Storm Water Drainage Wells

Interim Final



Source: Illinois EPA 1987

United States Environmental Protection Agency

Guidance on Storm Water Draina

Interim Final

May 1998

Office of Ground Water and Drinking Wate
Office of Water
U.S. Environmental Protection Agency

PREFACE

R0022206

This guidance was developed as part of the U.S. Environmental Protect for the management of Class V wells. This strategy was outlined in an A Register notice (pp. 44652 - 44668). The notice stated that "As a part o comprehensive management of Class V wells, the Agency will issue a t on the effective methods of managing storm water injection wells to ass Underground Sources of Drinking Water. The guidance will provide in are believed to provide adequate segregation from industrial process or techniques for minimizing the environmental impacts of injected storm intended to provide technical assistance on storm water drainage well n and local regulatory officials. Because this document is intended to ser rule-making, the guidance outlines suggested storm water management examples of storm water regulations and guidance from various States, regulatory requirements for construction and operation of storm water d

This guidance is designated as Interim Final. The reason for this design currently further assessing the impact of storm water drainage well prac sources of drinking water. This assessment of the environmental impact wells is being conducted as part of the Class V Study. Based on the resu EPA will determine whether storm water drainage wells need further re of the environment or if they can be safely operated through the use of Management Practices. When the assessment is completed, this guidan with modifications, depending on the findings of the assessment.

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Last Updated 5/98
http://www.epa.gov./r5water/storm/newstorm.html

Office of Ground Water and Drinking Water

United States Environmental Protection Agency

Guidance on Storm Water Drainage Wells

INTERIM FINAL

May 1998

6.0 STORM WATER DRAINAGE WELL DESIGN BEST MANAGEMENT PRACTICES (BMPs)

When an environmental agency is involved in siting of a storm water drainage well, it has an opportunity to recommend design features that minimize the risk of contaminating drinking water sources. Design features are often less expensive to install during construction than later as a retrofit. Responsible officials should ensure that the design and siting of the storm water drainage well minimizes the likelihood of both accidental and routine contamination. The following discussion of well designs and filtering systems is intended to provide a brief overview of the popular types of systems in use. For more detailed information on design considerations for storm water disposal systems, the reader is strongly encouraged to consult Design of Stormwater Filtering Systems (Claytong and Schueler, 1996).

The construction and siting measures recommended or required should be based on the type and magnitude of the contamination risk to ground water that a site's activities pose. For example, a site where a large number of vehicles are parked or travel through has a high probability that its storm water will contain dispersed oil, but a lower probability that some sort of spill or large release of petroleum product will occur. At such a site, a supervising agency may want to recommend oil/water removal devices in inlets passing storm water to the drainage well. In contrast, a residential site with few associated vehicles might not need such devices.

The following construction methods and devices can reduce the potential for pollution of ground water by storm water injection wells.

6.1 Sediment Removal

Sediment carried in storm water flows will enter a storm water drainage well, unless the well includes devices for removing that sediment. Sediment poses two problems: 1) it can clog the infiltration system causing it to fail; and 2) contaminants, including metals and phosphorus, can be attached to sediments, thus allowing for potential ground water contamination if sediments enter the infiltration system. In most cases the infiltration system will clog and cause an unplanned discharge to surface waters before a significant amount of contamination can be carried by sediment into the ground water.

Pretreatment methods used for preventing sediment from entering storm water infiltration devices include oil/grit separators, settling basins (catch basins or detention or retention basins), and filter strips and swales.

One of the chief difficulties with infiltration trenches is that they tend to clog with fine sediment, slowing the rate of infiltration into the soil. In many cases, the sediment enters the basin, trench, or dry well during construction of the facility. Measures to prevent sediment from entering the infiltration device include temporary diversions, such as sediment traps, roping off the trench area to prevent construction equipment or other traffic from compacting soil and stabilizing the area around the trench by planting vegetation. The trench or basin can also act a sediment trap during construction of surrounding buildings. After the site is fully stabilized, the site operator can remove the sediment and then excavate the remainder of the trench.

Oil/Grit Separators

Oil/grit separators, also called water quality inlets, consist of one or more chambers designed to allow sediments to settle out prior to entering the storm water well. Many separator designs also contain baffles so that the uppermost layer of water in each of the separators' chambers is retained. Material such as oil floating on top of the trapped water is retained and can be removed when the separator is cleaned. Sediment that is heavier than water will settle out at a rate determined by the density and size of the sediment particles. A typical separator design is shown in Exhibit 6-1. Separators slow inflowing water, allowing time for a portion of incoming sediment to settle out of the storm water. The portion of sediment that is removed is determined in part by the speed of water flowing through the separator, relative to the settling speed of the sediment and the depth of the separator. Separators are not very effective in removing particles and oil droplets that are entrained or dispersed within the flow as well as oil in the dissolved pahse. Similarly, lighter-than-water particles such as small oil droplets rise at a speed that depends on the droplet's size and density. When a separator retains water long enough to allow particles to settle or rise to the surface, it is effective at retaining these particles. If the holding time is too short, particles remain in suspension and are passed to the infiltration system. Separators are designed based on expected inflow rates and particle sizes. See Debo & Reese (1995) and Washington DOE (1992) for a more in-depth discussion of oil/grit separator design.

If not properly designed and frequently cleaned, separators allow trapped sediment to be resuspended and pass out of the separator during subsequent flow events. In summary, separators are not very effective and less desirable than settling basins or filter strips for removing sediment. Schueler (undated) showed for several locations that sediment retained in a separator in fact *decreased* over time, due to resuspension of sediment, rather than increasing as might have been expected (see Exhibit 6-2).

Advantages: Separators comprise a standard practice that is well understood and easily designed by most storm water engineers. They are relatively simple to construct, are available as pre-fabricated units or can be custom built from standard fittings, and are relatively easy to maintain.

R0022211

Disadvantages: Separators increase the cost of a storm water drainage well from several hundred dollars for simple catch basins to tens of thousands of dollars for complex multi-chamber installations. Separators require periodic cleaning and maintenance which may necessitate costly equipment such as a jet pump. Questionable effectiveness, particularly in separating dispersed petroleum products which depends greatly on the separator's design and its associated holding time.

Filter Strips and Swales

Filter strips and swales are vegetated buffers that trap sediment before it enters infiltration devices. Filter strips should be at least 20 feet wide. The width of the strip should be based on flow, site characteristics and pollutant loading. They should be graded to less than 2% so that water flows over them in sheets rather than as a concentrated stream. Sheet flow decreases the possibility of gully erosion and distributes contaminants over a wider area. Level spreaders such as slotted curbs may also be used to facilitate sheet flow. Vegetation also protects soil from being eroded. Roots and fauna in the soil also provide pore space for infiltration. Native vegetation should be used because it requires less maintenance, such as pesticides and fertilizers.

Catch Basin Inserts

Catch basins are often used to hold water before it flows to infiltration devices. Catch basin inserts can remove a large portion of oil, grease, and metals in runoff. The inserts consist of several filtration trays that hang down from the inlet grate. The top tray is an oil/grit separator, and the lower trays may be activated charcoal, which traps pesticides, fertilizers, and metals; reconstituted wood fiber, which traps oil and grease; or fiberglass insulation. Inserts should be replaced quarterly, and inspected more frequently during wet periods. While these inserts can remove potential contaminants, they require a high degree of inspection and maintenance. Additionally, they clog easily, preventing passage of storm water, and they are hard to remove without proper equiement.

Case Study 6.1: Municipal Building Infiltration Trench

6.2 Oil and Grease Removal

Petroleum products contain components such as benzene, which are known human carcinogens, that can potentially contaminate ground water. Petroleum can enter a storm water drainage well from (1) accidental spills, (2) intentional disposal of automotive products such as used motor oil (i.e., misuse of the drainage system), and (3) from oil residue washed from pavement. Oil and grit separators can remove contaminants before they enter infiltration facilities. Because of their expense, however, oil/grit separators are generally used only in relatively small, impervious areas that have a high potential for oily runoff (e.g., gas stations and industrial areas). Separators must be cleaned frequently (monthly to quarterly) to avoid clogging, or concentration and resuspension of contaminants. Furthermore, the oil and sediment removed from these devices may qualify as hazardous waste and should be tested prior to disposal.

Depending on their design, separators perform two functions; they (1) retain small

spills of floating liquids (e.g., spilled gasoline), and (2) allow some separation of floating materials and sediment from storm water.

Most separators consist of several chambers, or a single chamber with top and bottom baffles. Flow between the chambers is through outlets located below the water surface. This prevents floating oil from flowing to the next chamber. Dispersed oil will slowly rise in still water. If an oil/water separator is designed to allow sufficient holding time, oil droplets can also be removed from storm water. Generally there are two categories of oil separators, those designed to retain small spills, and those designed to provide extended holding time to allow separation of dispersed oil. A third type of separator uses "coalescing plates" made of polypropylene or fiberglass to separate dispersed oil. Debo and Reese (1995) describe equations for designing various types of oil/water separators.

In deciding where to recommend oil/grit separators, officials should weigh the added expense against the likelihood of well contamination caused by site activity. While petroleum contamination will vary from site to site, some land use related patterns have been noted by researchers. Schueler (undated) noted higher levels of total hydrocarbons in the water and sediment trapped in separators at gas stations and all day parking areas relative to street and residential areas (see Exhibits 6-3 and 6-4).

Spill Control Separators

Spill control separators are chambers that allow oil and grease to float to the top of a chamber, while water from below the oil layer is allowed to pass through to the storm water disposal system. They are effective at retaining small spills but do not remove dispersed oil droplets because they have a relatively short residence time (see Exhibit 6-5). These separators are essentially catch basins, but are designed to retain oil. They can often be included in a project at little additional expense above a simple catch basin.

API (American Petroleum Institute) Oil Separators

These separators are long vaults designed to retain storm water long enough for finely dispersed oil droplets to rise to the surface of the separator (see Exhibit 6-6). Design of these oil separation separators is discussed in API (1991). Because of their relative complexity, these types of separators should only be recommended where there is a relatively high likelihood of dispersed oil contamination. These specialized separators are not typically used for storm water treatment except where dispersed oil loading is high. If this is the case, an alternative strategy may be to minimize or eliminate the source of the oil.

Oil Absorbent Material

Oil absorbent pillows can be used to absorb petroleum products that are present in high concentrations; they do not absorb water. They can be left floating in an oil/grit separator and later removed during maintenance. One readily available model is roughly 18 inches long and absorbs up to 2 gallons of petroleum-based liquid. In combination with a grease and oil separator, hydrophobic pillows can minimize the amount of petroleum product passed on to an infiltration device. Because of their

relatively small capacity, they are not protective in a large-scale spill and are also not effective in removing dispersed oil. They are, however, a simple, inexpensive means of reducing oil contamination.

Coalescing Plates

Coalescing plates are sets of thin, closely-spaced sheets or plates designed to induce finely mixed oil to coalesce into larger droplets which are more easily separated from water. These plates typically are made from fiberglass or plastic. The primary advantage of this technology is that a separator can be sized smaller for a given application by more rapidly removing oil droplets from standing water (see Exhibit 6-7). Separators incorporating coalescing plates require periodic inspection and cleaning, and they can be expensive. Because of their relative complexity, these types of separators should only be recommended where there is a relatively high likelihood of dispersed oil contamination and trained staff available to perform proper maintenance. They are not routinely used for uncontaminated storm water.

Case Study 6.2: Oil/Water Separator Installation

Example State Guidelines

Below are examples of guidelines regarding oil and grease removal provided by several states. They are presented for use as a reference point for giving further guidance to owners and operators.

Example Requirements and Guidance

Arizona (Guidance) (Arizona DEQ 1995)

- Systems in general areas should include a hydrophobic petrochemical absorbent with a 128 ounce (1 gallon) capacity. Systems in heavy use/industrial areas should have an absorbent with a 256 ounce (2 gallon) capacity. Systems should use a shielding device to enhance separation of petrochemicals from water by gravity differentials.
- Wells should have a settling chamber with at least 1,000 gallons capacity.

Rhode Island (Requirements) (Rhode Island DEM 1994)

- All catch basins leading to an infiltration system must have a minimum sump depth of 4 feet and a nominal capacity of 850 gallons.
- All catch basin outlet pipes must have an elbow with a vertical pipe extending a minimum of 2 feet below the outlet pipe invert.

Washington (Guidance) (Washington DOE 1992)

- Oil separators must be sized for the 6-month recurring, 24-hour design storm. Larger storms must be diverted from the separators.
- Oil separators must have a front chamber or forebay with an area of at least 20 square feet per 10,000 square feet of area draining to the separator (87 square

feet per acre of drainage area).

Go to 7.0 Operational Best Management Practices (BMPs).

Go to 5.0 Well Siting Best Management Practices (BMPs) and Requirements.

Up One Level || Comments || Contacts || EPA Home

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Office of Ground Water and Drinking Water

United States Environmental Protection Agency

Guidance on Storm Water Drainage Wells

INTERIM FINAL

May 1998

Exhibit 6-3
Characterization of Pollutant Concentrations in the Water Column of Oil/Grit Separators - Effect on Land Use Condition

Sampled Parameter	All-Day Parking	Convenience Commercial	Gas Stations	Streets	Townhouse-Garden Apartments
N =	(8)	(6)	(7)	(6)	(6)
Ortho Phosphate Phosphorus (mg/l)	0.23	0.16	0.11	not detected	0.11
Total Phosphorus (mg/l)	0.30	0.50	0.53	0.06	0.19
Ammonia Nitrogen (mg/l)	0.20	1.58	0.11	0.19	0.20
Total Kjeldahl Nitrogen (mg/l)	1.18	4.94	2.5	.84	1.00
Oxidized Nitrogen (mg/l)	0.65	0.01	0.21	0.92	0.17
Total Organic Carbon (mg/l)	20.60	26.80	95.51	9.91	15.75
Total Hydrocarbons (mg/l)	15.40	10.93	21.97	2.86	2.38
Total Suspended Solids (mg/l)	4.74	5.70	no data	9.60	7.07
Extractable Cadmium (ug/l)	6.45	7.92*	15.29*	no data	no data

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Soluble Cadmium (ug/l)	3.40*	no data	6.34*	no data	10.34*
Extractable Chromium (ug/l)	5.37	13.85	17.63*	5.52*	no data
Soluble Chromium (ug/l)	no data	no data	6.40*	no data	4.79*
Extractable Copper (ug/l)	11.61	22.11	112.63	9.50*	3.62
Soluble Copper (ug/l)	8.22*	no data	25.64	no data	2.40
Extractable Lead (ug/l)	13.42	28.87	162.38	8.23	no data
Soluble Lead (ug/l)	8.10*	no data	26.90*	no data	no data
Extractable Zinc (ug/l)	190.00	201.00	554.00	92.00	no data
Soluble Zinc (ug/l)	106.70	43.70	471.00	69.00	59.00

Source: Schueler (undated)

Note: All reported data are mean values. Asterisks indicate that the mean is for observations in which the indicated parameter was actually detected.

Go to 6.0 Storm Water Drainage Best Management Practices (BMPs).

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http://www.epa.gov/r5water/storm/ex6-3.htm

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Office of Ground Water and Drinking Water

United States Environmental Protection Agency

Guidance on Storm Water Drainage Wells

INTERIM FINAL

May 1998

Exhibit 6-4
Characterization of the Quality of Trapped Sediments in OGS - Effect of Land Use

Parameter	All-Day Parking	Convenience Commercial	Gas Stations	Streets	Townhouse-Gar Apartments
N =	(8)	(6)	(7)	(6)	(6)
Total Kjeldahl Nitrogen (mg/kg)	1951.0	5528.0	3102.0	1719.0	1760.0
Total Phosphorus (mg/kg)	466.0	1020.0	1056.0	365.0	266.7
Total Organic Carbon (mg/kg)>/td>37915.0	55617.0	98071.0	33025.0	32392.0	
Total Hydrocarbons (mg/kg)	7114.0	7003.0	18155.0	3482.0	894.0
Cadmium (mg/kg)	13.2	17.1	35.6	13.6	13.5
Chromium (mg/kg)	258.0	233.0	350.0	291.0	323.0
Copper (mg/kg)	186.0	326.0	788.0	173.0	162.0
Lead (mg/kg)	309.0	677.0	1183.0	544.0	180.0
Zinc (mg/kg)	1580.0	4025.0	6785.0	1800.0	878.0

Source: Schueler (undated)

Go to 6.0 Storm Water Drainage Well Design Best Management Practices (BMPs).

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Re-Evaluating Stormwater

The Nine Mile Run Model for Restorative Redevelopment

Technical Appendix

Materials from the Nine Mile Run Design Charrette, October 1998

R0022219

Re-Evaluating Stormwater:
The Nine Mile Run Model for Restorative Redevelopment
TECHNICAL APPENDIX

October, 1999

This volume contains materials associated with a design charrette held October 14-17, 1998 in Wilkinsburg, Pennsylvania. That event examined possibilities for small-scale, site-specific retrofit and redevelopment activities that could simultaneously restore the hydrology of urban watersheds, contribute to management of stormwater runoff and sewer overflows, and revitalize the economic and cultural life of urban places. The charrette focused on the Nine Mile Run Watershed of Edgewood, Pittsburgh, Swissvale, and Wilkinsburg, as a model for development of physical and policy measures which could be replicated throughout the greater Pittsburgh region and beyond.

The main report: *Re-Evaluating Stormwater: The Nine Mile Run Model for Restorative Redevelopment*
by Bruce Ferguson, Richard Pinkham, and Timothy Collins
can be obtained from:

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This appendix was compiled and edited by Richard Pinkham, Rocky Mountain Institute.
Attribution of specific materials is given in each section of the appendix.

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ACKNOWLEDGMENTS

The project organizers gratefully acknowledge the generous funding provided to this project by The Heinz Endowments.

The following organizations provided considerable information and other assistance:

Allegheny County Sanitary Authority (ALCOSAN)
Allegheny County Health Department
Borough of Wilkinsburg
Borough of Edgewood
Borough of Swissvale
City of Pittsburgh
Pittsburgh Sewer and Water
Three Rivers Wet Weather Demonstration Program

Individuals who assisted development of the charrette with special perspectives and time include: Melisa Crawford, David French, Jan Oliver, Andrew McElwaine, Raymond Reaves, and John Schombert. Special thanks to the staff of the Hosanna House of Wilkinsburg for their assistance in hosting the charrette, and to Chris Leininger of Sustainable Home Design for preparation of site base maps.

We are grateful to EDAW, Inc. for providing staff experts for the charrette and design services for the main report.

We also acknowledge the inspiration and guidance of TreePeople, and its contractor, ReThink, Inc. Tree People has developed a similar model for urban restoration and water management in Los Angeles, beginning with the T. R. E. E. S. (Transagency Resources for Environmental and Economic Sustainability) charrette in Los Angeles, May 1997.

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Re-Evaluating Stormwater

The Nine Mile Run Model for Restorative Redevelopment Technical Appendix

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The following pages are a copy of the "briefing book" distributed to charrette participants prior to the October 1998 event. The brief provided background information and instructions to the teams.

ECOSYSTEMS AND INFRASTRUCTURE

NINE MILE RUN

Site-Specific Stormwater Management Options Charrette
October 14-17

Nine Mile Run Watershed
Pittsburgh PA

The Briefing Book

Rocky Mountain Institute
STUDIO for Creative Inquiry

Acknowledgments

We gratefully acknowledge the generous funding provided to this project by:

The Heinz Endowments

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Borough of Edgewood
Borough of Swissvale
City of Pittsburgh
Pittsburgh Sewer and Water
Three Rivers Wet Weather Demonstration Program

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We extend special thanks to the staff of the Hosanna House of Wilkinsburg for all their assistance in hosting the charrette, and to Chris Leininger of Sustainable Home Design for preparation of site base maps.

This charrette briefing book was written and edited by Richard Pinkham of Rocky Mountain Institute, and Tim Collins and John Stephen of the STUDIO for Creative Inquiry at Carnegie Mellon University, with assistance from Bruce Ferguson of the University of Georgia School of Environmental Design. Graphic design by Ema Tibbetts of Rocky Mountain Institute. Colleagues offering additional support at the sponsoring organizations included Bill Browning, Alexis Karolides, Michelle Sinsel, and Jen Uncapher at Rocky Mountain Institute, and Bob Bingham, Reiko Goto, Georgina King, Choli Lightfoot, Richard Pell, and Karin Tuxen at the STUDIO for Creative Inquiry.

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Charrette Schedule

Wednesday, October 14

8:00 PM

Informal reception, Holiday Inn Parkway West, 915 Brinton Road, Braddock Hills, Pennsylvania.

Thursday, October 15

8:00 AM

Arrive at Hosanna House, 807 Wallace Avenue, Wilkinsburg. Coffee served.

8:30 AM

Opening plenary session. Overview of charrette approach, objectives, and sites.

9:45 AM

Depart for sites by van. Four design teams go to respective sites; policy team tours all sites and key watershed features.

NOON

Brown-bag lunch provided at sites. Groups keep momentum going with discussions on-site over and after lunch.

EARLY PM

Return to Hosanna House. Team work time.

5:00 PM

Buffet dinner at Hosanna House.

6:30 PM

Plenary session: public invited.

7:15 PM

Break-out public "round-table" sessions. Teams discuss and exchange ideas, questions, and concerns with local citizens and officials.

8:30 PM

Round tables wrap-up. Teams continue discussions/design as desired. Hosanna House is open until 11:00 PM.

Friday, October 16

8:00 AM

Coffee served at Hosanna House.

8:30 AM

Team work time.

NOON

Working lunch. Teams prepare for reporting out.

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Friday, October 16, continued

1:00 PM

Plenary session—reporting out. Each team has 5 minutes to present core ideas, 5 minutes for Q & A.

2:00 PM

Plenary discussion of policy and institutional barriers and opportunities.

3:00 PM

Team work time.

7:00 PM

Buffet dinner at Hosanna House.

8:00 PM

Team meetings to assess progress towards required products. Continue design efforts as needed. Hosanna House is open until 11:00 PM.

Saturday, October 17

8:00 AM

Coffee served at Hosanna House.

8:30 AM

Team work time.

NOON

Working lunch.

1:00 PM

Final team work session. Finish products and develop presentation.

2:30 PM

Plenary session. Practice presentation sessions (5 minutes each plus 5 minutes critiques/suggestions).

3:30 PM

Closing plenary session: public invited. Overview of the charrette effort and brief team presentations.

5:30 PM

Reconvene and close.

5:45 PM

Reception at Hosanna House. Refreshments served.

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Charrette Teams

* Invited but not confirmed as of publication.
Team assignments are subject to change.

Charrette Facilitator: Bruce Ferguson. Landscape Architect; University of Georgia; Athens, GA.
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Additional graphics support provided by students from Carnegie Mellon University Department of Art, Carnegie Mellon University Department of Architecture, Slippery Rock University Sustainable Systems Program, and Pennsylvania State University Department of Landscape Architecture.

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About this Charrette Briefing Book

This document is your guide to the topics, approach, objectives, and required products of this charrette. Following sections of this brief introduce the purposes and objectives of the charrette, and describe the region and the watershed in more detail, including the problems being experienced, current efforts to address those problems, and the broader context of regional policies and programs that should inform the work of charrette participants. The overall design objectives and guidelines for the teams are described next, followed by the specific work products required of the teams. Each site is then described in detail, and some directives specific to individual teams given.

A storm sewer outfall along the channelized portion of Nine Mile Run.

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Introduction

The greater Pittsburgh region faces a host of problems attributed to wet weather events: combined sewer overflows (CSOs), sanitary sewer overflows (SSOs), and erosion and pollution of stream channels and rivers from stormwater flows. Sewer separations, new detention facilities, and new or expanded treatment facilities to reduce sewer overflows and stormwater runoff problems could run up a huge bill for the region.

The organizers of this charrette believe that wet weather solutions are an opportunity to reconsider the form and function of infrastructure within the context of urban ecosystems. Over the long term, the built environment has replaced much of the natural environment in form while subsuming natural hydrologic function. Short-term problem solving has created an urban infrastructure which has been the focus of iterative single-purpose approaches which traditionally address the downstream effect rather than the upstream cause of problems. Municipal managers are now faced with two issues regarding the existing sewer and stormwater infrastructure: it often no longer functions to its original design specifications, and it is unable to meet contemporary regulatory guidelines and societal needs.

Our thesis is simple: rather than just retrofit the conventional system of conveyance and treatment, retrofit to mitigate the quantity and quality of material being conveyed. To achieve this, a whole-systems perspective is required—one that examines and manages flows throughout the entire urban system. It is essential, then, to embrace two points: a) manage precipitation as close to where it falls as is physically and economically feasible, and b) use natural processes to advantage. Urban infrastructures can reclaim eco-system functionality by utilizing the water purification services offered by vegetation and microorganisms, and the water storage capacities of soils and subsoils. To do so, society can use a range of measures to minimize, use, re-use, infiltrate, treat, or detain precipitation at the level of individual sites and neighborhoods. The techniques for doing so have been proven in new development applications in the U.S., and are being aggressively explored for urban retrofit of stormwater systems in Europe and Japan. Since there are few models of similar urban programs in the U.S., the goal of site-based, adaptive urban design for Allegheny County communities may provide an important model for the region and maybe the country.

We hypothesize that:

- site-based retrofits and urban redevelopment for improved wet weather management can be technically and economically feasible, and
- site-based retrofit strategies can improve the value and livability of the built environment.

We will test this hypothesis in an intensive, three-day design charrette by focusing teams on different sites in the Nine Mile Run watershed, a 6.5 square mile, highly urbanized watershed tributary to the Monongahela River. Nine Mile Run suffers from CSOs, SSOs, runoff problems, localized flooding from drainage difficulties, and depleted stream base flow due to reduced ground water recharge. The watershed is a typical older urban area, with both positive and negative attributes. It has its share of traffic congestion, automobile dependency, air quality problems, solidly built homes, strong neighborhood character and identify, and mosaics of neighborhoods that are rich and poor. How improved wet weather management can integrate with and improve the larger urban fabric is an important topic for this charrette.

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Charrette Purposes

- To expand the ideas informing decision-making on wet weather management in the Nine Mile Run Watershed specifically and Allegheny County generally.
- To encourage a long-term shift in values, connecting ecosystems to infrastructure within a new paradigm of sustainability defined in terms of quality of life, and long-term environmental-economic benefit for cities.

Charrette Focus

- This charrette focuses on retrofit and redevelopment opportunities at the scale of individual properties and city blocks, illustrating possibilities for solving watershed problems at the source, in the urban landscape where the rain falls.

Charrette Objectives

- To develop conceptual designs and water management practices illustrating specific, effective techniques for local sites.
- To outline the economics of mitigating stormwater quantity and quality close to the source.
- To show how site-based stormwater management techniques can assist achievement of other local and regional objectives for improved urban function and quality of life.
- To enable communities to move towards implementing these measures by identifying information needs and essential subsequent design steps.
- To outline policies, management structures, and programmatic opportunities to take advantage of site-based strategies.
- To educate the public and decision makers about site-based wet weather management options, and generate interest and excitement for these ideas.

Charrette Structure

Innovative solutions to wet weather problems require integration of multiple perspectives and types of expertise, including art, landscape architecture, architecture, engineering, soils and hydrology, public works, planning, and more. By pulling together persons with diverse knowledge into design and policy teams in an intensive, results-oriented forum, this charrette hopes to encourage whole-systems thinking and quickly turn that thinking into design ideas. The teams include local individuals with relevant expertise or responsibilities, nationally recognized experts in site-based stormwater management, and design students to help illustrate concepts generated by each group. This mix serves several purposes: introducing new ideas and perspectives into the community, grounding design in knowledge of local conditions, and validating good ideas already present in the community.

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Four teams will address four different sites representative of typical land uses and geographic conditions in the watershed and region. Each team will be given specific design guidelines to meet for improved water management and ancillary benefits.

"Regent Square Gateway." Public rights-of-way and a former supermarket on cut and fill lying at the junction between the developed portion of the watershed and the open space of the lower watershed.

"Sterrett School." A middle school building and associated grounds, in a block with several single-family residences, on alluvial substrates at mid-elevation in the watershed.

"Edgewood Transit Crossings." A busy street intersection, associated small storefront buildings and residences and a church, bounded by a railroad bed soon to become a busway. Lies on deep alluvial substrates typical of the middle portion of the watershed.

"Hunter Park." Municipal playing fields surrounded by low-income homes, on shallow soils and hillsides of the upper watershed.

A fifth "policy" team will address implementation opportunities and barriers in relevant local codes, regulations, plans, policies, infrastructure programs, etc. This team is charged with outlining an effective integrated watershed management framework, and with identifying ways to improve implementation prospects for the sorts of physical approaches identified by the site teams.

Facilitation and "reporting out" from the teams will encourage cross-fertilization between the design teams and inform the policy team of measures and issues identified by the designers. All teams will participate in an open house on the first evening, which includes plenary and break-out sessions to give local citizens an opportunity to express issues and concerns to any of the teams, and a chance for each team to get feedback on initial designs and concepts. A second open house late Saturday afternoon includes brief presentations from each team, followed by break-outs for evaluation and discussion between each team and community members.

"When Patrick Geddes coined the word Eutopia, meaning "good place," in his address to the Sociological Society in July 1904, and compared it with the commonly understood Utopia coined by Thomas More, a word derived from the Greek "no place," he summed up a fundamental tenet of the regional imperative: that it makes sense to design with the forms and cultural and ecological processes already present in a location rather than to force an idealized, preconceived plan upon a site. Eutopia is assured when culture and ecology become part of design. Utopia is the consequence of ignoring them."

—Michael Hough. Out of Place, Restoring Identity to the Regional Landscape.

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Issues and Efforts

Regional Wet Weather Problems

Western Pennsylvania faces significant wet weather management problems. During 1997, the Allegheny County Health Department issued health advisories on 45 days of the recreational boating season urging residents to avoid contact with river water. The need for these warnings traces to wet weather sewage overflows and sewage system bypassing occurring in combined sewer systems and in sanitary sewer systems. Besides the health threats posed by such discharges, they compromise attainment of "fishable and swimmable" objectives of the federal Water Pollution Control Act of 1972 and subsequent standards of the federal Clean Water Act Amendments.

Combined sewer overflows (CSOs) occur at structures designed to release mixed sewer and stormwater when flows in combined sewer lines exceed system capacity. They are legally permitable under the National Pollution Discharge Elimination System, but must be reduced. The regional sanitation provider, Allegheny County Sanitary Authority (ALCOSAN), is due to release its Long Term CSO Control Plan in late 1998 or early 1999.

Sanitary sewer overflows can result from illegal connections of roof and basement drains and infiltration into cracked or disjointed sanitary sewer lines. Excess flows leak out through cracks or by pushing off manhole covers as pressures mount during wet weather. In addition, when the ALCOSAN regional system was designed in the 1940s, dedicated overflow structures were built into sanitary as well as combined sewer lines, because older urban sanitary sewers were known and expected to include many roof and basement drain connections. Federal and state regulatory agencies now consider SSOs illegal. The EPA and the U.S. Department of Justice are currently considering litigation or administrative actions against 51 communities and ALCOSAN to eliminate SSOs. These agencies may level penalties in excess of $275 million against the targeted entities.[1] Corrective actions to eliminate the overflows will cost much more.

Responses to the SSO problem have been many. A number of communities have built or are considering water storage tanks and detention basins to hold excess wet weather flows. Projects to rehabilitate cracked sanitary sewer lines or replace those lines altogether are underway or in the works in several municipalities.

Recently the Allegheny County Sanitary Authority and the Allegheny County Health Department joined forces to establish the Three Rivers Wet Weather Demonstration Program. Using federal, local, and in kind funding, this eight-year program will emphasize watershed-based approaches, establish a wet weather management planning process, and award funds to competitively selected projects demonstrating innovative technical, institutional, and financial mechanisms to control sanitary sewer overflows. The program will soon issue RFPs to local municipalities for the first round of competitive demonstration grants. Source reduction techniques will be an important emphasis of the program.

As in many urban regions, surface runoff from storms is also a local problem, causing localized flooding and erosion of stream channels. Pennsylvania's Storm Water Management Act of 1978 (Act 167) requires all counties to develop stormwater management plans for the watersheds in their boundaries. These plans must set forth provisions to ensure that development does not alter stormwater runoff quantities in ways that adversely affect health, safety, and property. At the current time, plans have been developed and adopted by Allegheny County for most but not all of the watersheds in the county. Act 167 plans only regulate new development and redevelopment; they do not address remediation of problems from the existing pattern of development.

A frequent corollary in urban areas to increased wet weather runoff is reduced dry weather stream flow. As impervious surfaces speed the flow of precipitation toward stream channels, they prevent the infiltration of water into the soil, reducing recharge of the ground water that supports the base flow of streams long after the most recent rains. Thus, the urban landscape produces both "higher highs" and "lower lows" compared to pre-development hydrology. Lower base flows reduce the quantity of instream habitat, and often result in elevated stream temperatures and reduced oxygenation of the water. These problems have been noted in several Pittsburgh area streams.

Recently, federal regulatory interest in nonpoint source pollution from urban runoff has increased. Urban stormwater runoff carries pet feces, lawn fertilizer and pesticides, oils and greases, trash, particles sloughed from automobile brakes, and pollutants deposited on the city from the air. The U.S. Environmental Protection Agency considers this nonpoint source pollution problem an important national priority, and has recently promulgated new stormwater quality regulations for smaller communities and properties to augment requirements already on the books for larger communities and developments. Sustainability and regulatory actions demand that the Pittsburgh area address the quality as well as the quantity of its stormwater runoff.

Nine Mile Run

The Nine Mile Run Watershed and Its Issues

Nine Mile Run (NMR) is an urban stream draining a 6.5 square mile watershed. The upper portions of the watershed are highly urbanized, while the lower portions are undeveloped. The lower, open portion of the Run is under 1.8 miles. It is a third order creek, which in turn drains into the Monogahela River. The larger of the two second order tributaries has been culverted. It winds underneath three municipalities to emerge in an open channel in Frick Park (400 acres) before meeting the other major tributary which drains Frick Park, the Homewood cemetery and surrounding streets. The watershed's topography is characterized by a lower ravine, a mid-level plateau, and surrounding small hills above.

The Nine Mile Run stream ecosystem is plagued by two water quantity issues: scouring and erosive flows during storm events, and diminished flow during the dry season. The watershed is 34 percent open space. Runoff and springs within Frick Park and Homewood Cemetery produce three small (first order) creeks that exhibit good water quality and a diversity of aquatic organisms. Despite this fact, Frick Park and Homewood Cemetery place a significant amount of surface flow into storm sewers to protect trails and lawns. A study of Frick park during construction of the sewers in 1947 indicated a significant drop in the ground water levels and a cause and effect on the plant life.[2] NMR has lost its floodplain and wetlands to industry, highway construction, and pressing recreational uses. Because of this, NMR digs into its streambed with a powerful erosive force. The effect of this includes a sediment load that is detrimental to life in the stream; it also has an obvious effect on the Monongahela, as illustrated by the sandbar that has developed at the mouth of NMR. The ecosystem is further stressed by low flows in mid-summer, resulting in lower oxygen content and significant increases in water temperature.

Water quality in Nine Mile Run is negatively impacted by inputs of domestic sewage from unauthorized sewer discharges to the culverted section of NMR. Sources include sewer leakage or unauthorized sewer discharges to stormwater sewers, and sanitary sewer overflows

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(SSOs) and combined sewer overflows (CSOs) to NMR along its length. These sewage inputs are contributed by all four watershed communities and have been occurring for many years. The sewage inputs cause high levels of fecal coliform bacteria in the stream during both dry and wet weather conditions, indicating the water is unsafe for human contact. This is recognized as a significant human health hazard by the Allegheny County Health Department.

The NMR streambed will soon be surrounded by a contiguous public space from where its first tributary emerges from the culvert at old Braddock Road right to its mouth at the Monongahela. Stream access will be provided to users of an expanding regional greenway system. There are three tiers of problems that need to be addressed: (1) the value of NMR is lost to most viewers upon seeing the trash, smelling the SSO/CSO discharge points, and observing the detritus of sewer, highway, and urban neglect which defines the stream and floodplain; (2) fecal counts in excess of EPA/Pennsylvania Department of Environmental Protection standards for human access and use occur on this stream 365 days a year; (3) stormwater events are extremely dynamic resulting in a torrent minutes after a major rain event. This can present a potential danger to anyone in the stream channel. Storm events are also laden with fecal matter, a problem which is illustrated by chronically discharging manholes.

The outfall of the Wilkinsburg culvert at old Braddock Road at low flow.

...and after a rainstorm.

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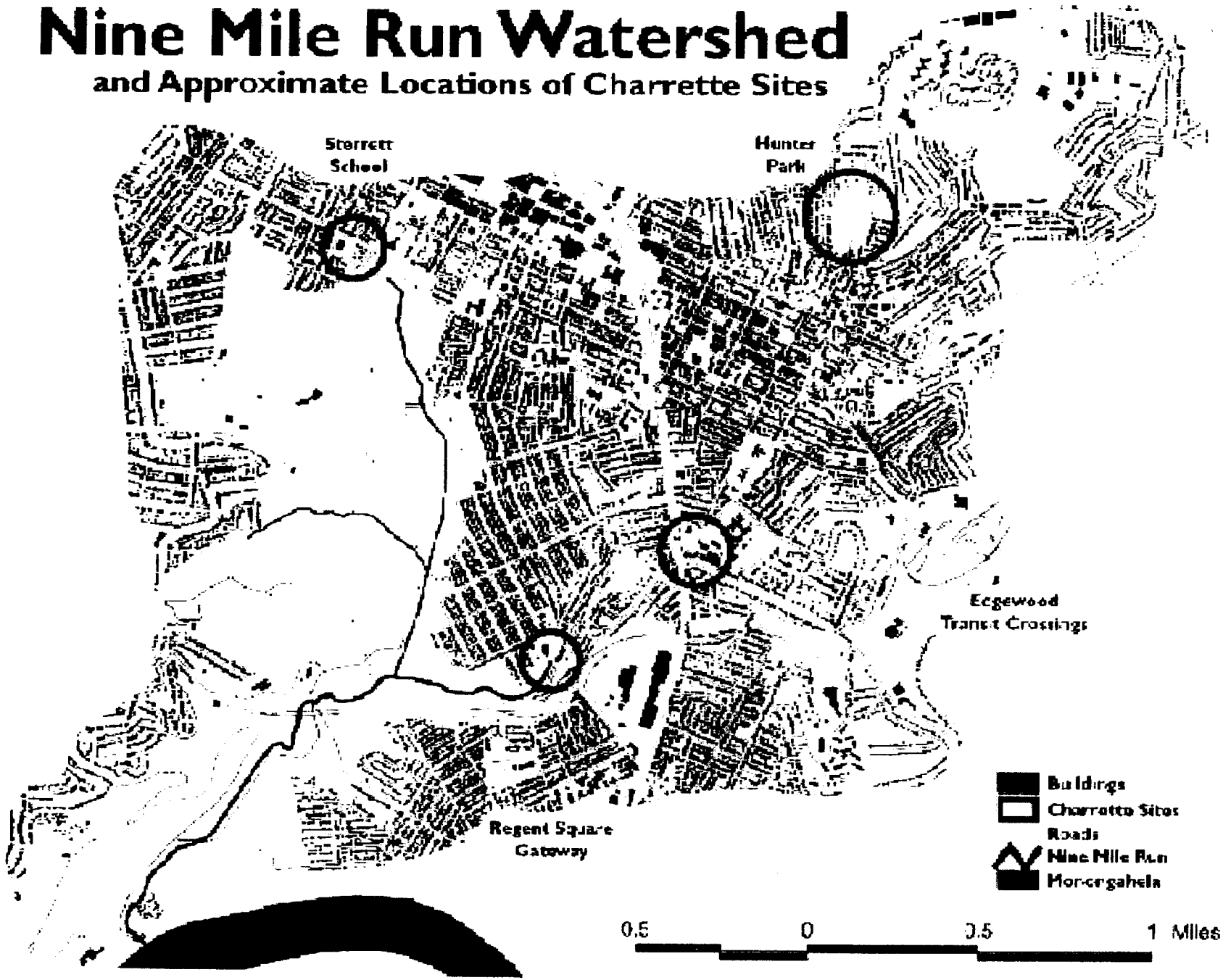
Nine Mile Run Watershed

and Approximate Locations of Charrette Sites



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Current and Proposed Nine Mile Run Improvement Projects

A number of recent, current, and proposed efforts are designed to improve the value of the Nine Mile Run greenway corridor:

- A River Conservation Management Plan, funded through the Pennsylvania Department of Conservation and Natural Resources and organized by the Pittsburgh Department of City Planning with the STUDIO for Creative Inquiry acting as a consultant, recommends a course of action for the restoration and enhancement of the watershed; analyzes the stream and its corridor, including flow, chemical composition, assessment of existing data on wastewater and stormwater, fauna and floral diversity; and evaluates management options that encourage watershed-based approaches for Nine Mile Run and the greenway.[3]
- An ecological assessment and alternative planning program for the ecosystem restoration of the lower watershed, managed by the STUDIO for Creative Inquiry and funded by the Heinz Endowments, includes landscape design alternatives informed by interdisciplinary analysis. The process relies on the expertise of an affiliated team of experts, and the design options are refined by watershed stakeholders during community meetings.[4]
- The City of Pittsburgh has submitted a letter of intent to the U.S. Army Corps of Engineers and has developed a preliminary habitat restoration plan, seeking funding under Section 206 of the Water Resources Development Act of 1996 for aquatic ecosystem restoration. The habitat restoration approach for this project could include: in-channel physical restoration of the aquatic habitat that has been damaged by storm flows; reductions in storm flow velocities and treatment of stormwater through diversion/detention and wetland development; and restoration of dry weather water quality.[5]
- The City of Pittsburgh has secured funding through the U.S. EPA's Sustainable Development Challenge Grant program to investigate techniques for revegetating a steel slag dump site in the lower watershed that is being developed as a sustainable urban community and public open space. This project seeks to find innovative solutions to a range of site remediation challenges on steep slopes, and to develop a protocol for creating an economically viable and aesthetically pleasing succession-based revegetation program for slag slopes.
- The watershed sewer system needs a thorough inspection to identify broken and blocked piping, sewer system leaks and illicit connections, and sanitary sewer connections to the stream culverts. This maintenance could be accomplished with flow monitoring, video inspection, and mapping. Local municipalities have begun some of this work, which could lead to rehabilitation programs and joint use agreements to abandon older, parallel trunk lines in favor of those in better condition.

There is an emerging recognition that the ultimate success of efforts in the lower watershed depends substantially on reduction of storm flows and sewer overflows that emanate from the urbanized upper watershed. Nine Mile Run restoration proponents increasingly believe integration of infrastructures and ecosystems is required not just in the open space of the lower watershed, but in and among the buildings, parking lots, streets, parks, and yards of the headwaters.

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Context: Regional Efforts for Sustainability

Between 1980 and 1990, the population density of the Pittsburgh region (excluding Butler County) decreased by 7.3 percent, but the urban/suburban portion of the region actually increased in area by 9.1%, absorbing farmland and open space.[6] The trend is similar throughout Pennsylvania. In Pennsylvania's ten largest metropolitan regions the population grew by only 13% between 1970 and 1990, whereas land area consumed grew by 80%.[7]

The 21st Century Commission[8] has identified that land use decisions and infrastructure are interrelated. The Report states that "Pennsylvania must maintain and expand, where appropriate, an infrastructure that promotes and enhances the efficient use of land."[9] . A number of environmental efforts in Pittsburgh relate to the effort to counteract this dispersionary trend by designing a more attractive urban fabric.

The Department of City Planning is undertaking a master planning process for its four regional parks, including Frick Park. The planning initiative is assisted by a newly created private sector non-profit, the Pittsburgh Parks Conservancy. The Conservancy, modeled after New York City's Central Park Conservancy, is intended to facilitate private sector funding and enable volunteer stewards for the City's four regional parks. Another open space planning and visioning effort, focused on neighbor parks and greenways, was recently administered by the Pittsburgh Community Design Center. A Vision Paper, developed through a series of public meetings and a design charrette, will be used to define potential demonstration projects, build a broader constituency, and market concepts to a wider audience.

Trails and bikeways are playing an important role in reconnecting citizens to the natural environment that surrounds them. The past ten years have seen an exponential growth in bicycle and walking trail development in the region, resulting in large part from the interest in the conversion of rails to trails. The Allegheny Trail Alliance, a coalition of the seven volunteer trail groups, is working to bring about a motor-free connection between Pittsburgh and Washington D.C. Locally, the plans for the link include a trail on the North Shore of the Monongahela River, across the mouth of Nine Mile Run, with connections through the Nine Mile Run Greenway, adjacent to the stream, to the Old Braddock Road site. The Port Authority of Allegheny County's preparations to provide new mass transit infrastructure for Pittsburgh's eastern suburbs provide another linear transportation opportunity. The Port Authority is designing to extend an existing busway along a rail line through Wilkinsburg, Edgewood, and Swissvale. The communities are not entirely pleased with the design and the planning process. Regardless, the project is mass transit that could reduce auto use and could provide bike lanes along the right-of-way.

Collaborative efforts are also advancing regional watershed agendas. For instance, the Allegheny Watershed Network, a project of the Pennsylvania Environmental Council, was formed to allow citizen groups and government agencies active within the Allegheny River watershed an opportunity to exchange information and ideas about the protection and enhancement of their local water resources. The Pennsylvania Department of Conservation and Natural Resources is using its River Conservation Program to encourage greater collaboration on a watershed basis. In addition to the Nine Mile Run Greenway Project, the DCNR has funded a plan for the thirty-five miles of riverbank within the City of Pittsburgh, planning for the Monongahela River upstream from Nine Mile Run, and planning for the Montour Run watershed in western Allegheny County.

The Montour Run plan is for a watershed impacted by large scale office park, big box retail development, and the large regional airport. The Airport Corridor was the subject of an

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extensive charrette process led by the Pittsburgh Chapter of the American Institute of Architects. The resulting report, *Reshaping the Region*, reveals large scale plans which integrate ecology with commercial and residential development. The integrated examples strive for a high quality of living. The community-informed design process led to a set of recommended principles, including:

- Design streets, pedestrian paths and bike paths as a continuous network of fully connected systems that connect housing to workplaces to shopping districts to parks and public places;
- Include sustainable water management, plant habitat and wildlife habitat management as part of any overall community infrastructure plans.[10]

Efforts to encourage community dialogue about land use, infrastructure, and the environment on a regional basis are being encouraged by the Environmental City Network through the Sustainable Pittsburgh Campaign. Sustainable Pittsburgh was created in part as a response to the President's Council on Sustainable Development and their September 1998 visit to Pittsburgh. Leaders in this effort hope that through community action, the region will emerge as a model or demonstration site for initiatives on sustainable development, community action, and multi-sector partnerships.

"What does this land help us to do, require us to do, permit us to do?"
—Wendell Berry

Toward increasing public awareness to environmental and quality of life issues, the Environmental City Network is facilitating development of the Pittsburgh Green Map to visually represent the region's landscape and environmental indicators. The mapping project will depict environmentally relevant information for publication and dissemination to the general public through computer networks and hard copy publications.

Western Pennsylvania's industrial heritage, with life in the region still dominated by industry, ethnic tradition, and communities, is the subject of regional development efforts as well. Individual projects are being pulled together by the Rivers of Steel Management Plan, coordinated by the Steel Industry Heritage Corporation, which seeks to preserve and celebrate the story of Big Steel and its related industries. The Management Action Plan served as the basis for the establishment of the Rivers of Steel National Heritage Area in 1996 (Public Law 104-333).

The Green Building Alliance is a local non-profit formed to facilitate the cost-effective and integrated use of environmentally responsible and technologically advanced site development and building design, construction and operation practices. This organization is implementing education, technical assistance, research, and development projects in the greater Pittsburgh region to help create more livable places for current and future generations.

Energy reduction and green practices are also brought out to the Pittsburgh community through the Green Neighborhood Initiative, a project of Conservation Consultants, Inc. The Initiative and its partners, the Western Pennsylvania Conservancy and the Pennsylvania Environmental Council, provide residential and commercial energy audits, community greening, and environmental education opportunities to targeted urban neighborhoods. The underlying mission is to work with community groups and volunteers to integrate intensive environmental services into existing neighborhood traditions.

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Design Guidelines

I. General Approach

This section of the brief specifically directs the site design teams, and also orients and informs the policy team. Your focus throughout the charrette should be on site-based retrofit strategies to infiltrate, detain, and treat stormwater. While off-site approaches exist, the charrette organizers believe that on-site retrofit technologies and practices should be a major focus of solutions to wet weather problems in the region. Our premise is that broad application of on-site strategies to other sites throughout the watershed and region can produce significant cumulative stormwater management benefits. Your job is to identify and illustrate the possibilities and benefits of on-site approaches.

The sites in this charrette are roughly a city block in size. We have defined sites at this scale to allow and encourage consideration of measures that can be taken at the level of individual properties *and* measures that are best integrated with neighboring properties or local public streets and rights-of-way.

It is possible to look at wet weather management two ways: as just a water management activity, or as part and parcel of the multifaceted, multi-functional urban fabric—a function deeply embedded in that fabric, simultaneously affecting and affected by many other urban activities. We challenge you to adopt and illustrate in this charrette the latter, broader approach—to identify on-site techniques to manage stormwater while simultaneously maintaining and improving other urban functions and the livability and sustainability of these sites and the region.

Stormwater management is but one function of urban infrastructure, but it touches on multiple other functions, for better or for worse depending on how it is done. "End of the pipe" solutions are not problematic per se. Rather, they often fail to attack a problem at its source, or they are single solutions to single problems. We believe that sustainability, long-term effectiveness, and optimal economic performance require approaches that go to the root of problems *and* satisfy more than one objective at a time.

Some site-based stormwater management retrofits may be justified within the narrow economics of cost comparisons between conventional and alternative projects. They can be chosen and implemented now, on those terms alone. Other retrofits may be justified by pointing to broader economic or non-economic benefits besides stormwater management. Many can and should be included in redevelopment projects and the myriad small adjustments to public and private properties that will over the long term transform the urban fabric into a better functioning, more sustainable, and more liveable landscape if adequately and holistically conceived and implemented.

II. Runoff Management

A. *Aim to infiltrate or detain (in that order of preference) the runoff from a two year, 24 hour storm (2.50 inches) on-site.*

Rationale: The two year, 24 hour storm is one of the design storm thresholds specified in the Monongahela River Watershed Stormwater Management Plan.[11] While this plan focuses on new development and the goals and hydrology of retrofits are rather different, the two year, 24 hour storm is a commonly understood yardstick for stormwater management. Meeting this performance threshold will allow others to easily understand the wet weather performance of your designs. It is an ambitious goal for retrofitting these highly urban sites. Larger storms are rare events: they constitute only a small percentage of total urban runoff, and most hydrologists agree that precipitation

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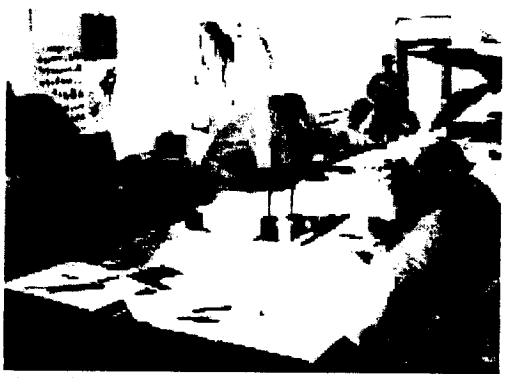
events of lesser quantity and greater frequency cause most of the water quality and channel degradation problems associated with urban runoff. Placing the priority on infiltration is consistent with the Monongahela River Watershed Stormwater Management Plan, with the need to reduce the volume as well as the peak flow of runoff to sewers and stream channels, and with restoration of a more natural watershed hydrologic regime, including recharge of ground water to support base flows.

B. *If you cannot infiltrate or detain the 2 year design storm on-site, design as much hydraulic capacity as the site allows, and indicate in your narrative the site-specific constraints that limit on-site hydraulic capacity.*
Rationale: In retrofit situations such as Nine Mile Run, with its many wet weather problems, any water removed from immediate runoff into combined or sanitary sewer lines, stormwater lines, or stream channels is a benefit. Retrofits should be encouraged even when they do not meet thresholds based on pre-development hydrology—any improvement is better than none. A comprehensive retrofit strategy for the watershed and the region should allow and enable any level of improvement a specific site allows, consistent with cost-effectiveness guidelines.


Typical charrette activities.

C. *Treat stormwater on-site for improved water quality as needed per your judgment of need and efficacy.*
Rationale: Some runoff sources (e.g. rooftop runoff) have less need for water quality treatment than others (e.g. parking lots). Some infiltration techniques may have sufficient treatment "built-in." The soil and subsoil is often a very effective treatment system.

D. *Provide for overflow to combined sewer lines or stormwater lines or stream channels when the capacity of your on-site management system is exceeded.*
Rationale: This is standard and necessary practice for on-site measures. The conveyance infrastructure is largely in place (albeit in need of rehabilitation in many locations).

E. *Address local drainage problems as appropriate and possible, and exercise care that your retrofits do not create other problems, such as flooding basements or undermining road base.*
Rationale: Like many urban sites, some of the sites identified for this charrette exemplify additional urban runoff problems beyond CSOs, SSOs, and stream channel degradation. These include localized street flooding (in part caused by off-site runoff) and basement flooding. Clearly, on-site strategies should not exacerbate these problems, and should contribute to their solution or mitigation wherever possible.

F. *Assume that whatever is physically possible is or will be institutionally possible.*
Rationale: It is very easy to point to institutional reasons why on-site approaches "cannot be done." There are barriers in codes, regulations, and policies at many levels of government. Programs and authorities to fund, implement, and maintain these measures are lacking. And so on. The job of the design teams is to illustrate what is physically possible. The policy team will address the institutional barriers and recommend

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solutions and opportunities to enable on-site strategies. Design teams should make note of potential institutional barriers and opportunities and pass these notes on to the policy team. We have scheduled a plenary discussion at the charrette mid-point as one opportunity to share concerns and ideas on policy and institutional issues.

III. Integration of Functions and Benefits

A. *As you retrofit your site for improved stormwater management, maintain or improve other urban infrastructures, functions and amenities. Look for synergies in the following areas (this list is illustrative; you are not bound by or limited to it):*

- *Water supply.* Harvest runoff for watering of landscapes, car washing, or other purposes.
- *Green waste management.* Utilize chipped or composted vegetative waste for soil amendments to improve infiltration, or as filter media.
- *Pedestrian access.* Reduce dependence on automobiles and replace automobile "habitat" with softer, more permeable spaces by rehabilitating sidewalks, creating buffers from moving cars, etc. Add new pedestrian connections along "desire lines" to unify the site and related parts of the community without the need for motor vehicles. Improve foot and bicycle access to transit systems.
- *Traffic calming.* Narrow neighborhood streets and create permeable parking lanes or bays.
- *Energy Use.* Shade buildings and capture/evaporate precipitation with increased tree canopy cover. Reduce the site-specific and regional urban heat island effect by minimizing paved surfaces.
- *Air quality.* Reduce air pollution by decreasing automotive dependence, and capture/filter pollutants with vegetative cover.
- *Employment.* Use the multiple benefit streams of dispersed stormwater management measures to create and sustain jobs in their construction and maintenance.
- *Habitat protection and creation.* Install vegetated swales, daylight previously culverted drainages, create water gardens and bioretention zones, and otherwise increase the diversity and abundance of ecological niches in the urban environment.
- *Recreation and leisure.* Create trails, pocket parks and other amenities that increase perviousness and provide for active or contemplative diversions on-site, or improve access to nearby amenities.
- *Beautification.* Use the "softer, greener" nature of many on-site stormwater management measures to aesthetic advantage.
- *Economic and social development.* Improve neighborhood vitality and property values through the above approaches and other interrelated strategies.

  Rationale: As stated above, we believe that sustainability, long-term effectiveness, and optimal economic performance require approaches that satisfy more than one objective at a time. At the present time and for the foreseeable future, the affordability of single-function infrastructures is in doubt. Broad benefits imply broad constituencies, thereby increasing the fundability of construction, and the motivations and potential funding sources for maintenance.

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IV. Budget

A. *Retrofit your site to manage stormwater on-site within a budget of $2.00 per gallon ($15.00 per cubic foot) of hydraulic capacity for on-site infiltration, detention, or treatment.*
Rationale: Water removed from immediate runoff into combined or sanitary sewer lines, stormwater lines, or stream channels is water that does not have to be detained downstream, and may not require treatment as well. Costs for conventional detention tanks and basins recently built or proposed for sewage system bypass reduction in the greater Pittsburgh region have ranged from $0.88 to $3.31 per gallon of total capacity, for construction costs only, with most facilities costing between $1.00 and $2.00 per gallon.[12] Design, land acquisition, legal, maintenance, and replacement costs, operational capacities, and many other considerations would have to be figured into a rigorous avoided cost analysis. Costs of treating the detained runoff to the applicable water quality standards would also have to be included. Additional "hard" or "conventional" infrastructure approaches are also available and may have greater or lesser costs. Nonetheless, these figures provide an initial, rough guide to the potential value of managing stormwater on-site. We suggest that $2.00 per gallon of hydraulic capacity is a reasonable "ballpark" figure for the value of water managed on-site. If society is prepared to fund downstream wet weather management at this level, it should be willing to invest similarly in alternative approaches accomplishing similar or complementary results on-site.

A combined sewer overflow structure along Nine Mile Run.

We realize that precise costing of your retrofits is not possible in the available time. We trust that your experience and judgment as designers will allow you to assess the rough magnitude of potential costs and calibrate your designs accordingly. Successful design processes usually involve an iterative exchange between physical possibilities and economic realities. We offer this budget as a guide to the "first iteration" this charrette aims to illustrate.

Note that this "budget" can be applied in different ways: as a total budget for one or a combination of retrofits, or as an incremental budget above costs that would be covered for other reasons. Some retrofits can or must "pay their own way" based on their runoff mitigation benefits alone. Others may be cost-shared with other projects and functions. As an example of cost-sharing, when property owners or municipalities replace pavements that are deteriorated or torn up for utility work, the cost of this replacement is borne by the owner for reasons other than stormwater management. Your budget can fund the incremental cost of simultaneously installing measures that increase on-site hydraulic capacity or water quality treatment—your budget in this case does not have to fund the entire cost of the pavement replacement project. Please identify and take advantage of cost-sharing opportunities. Make note of these opportunities in your narrative report, and indicate if they are likely to occur in the near term or long term.

B. *Should you find it necessary or desirable to exceed this hydraulic capacity budget, indicate in your narrative your reasons for doing so.*
Rationale: Some of the potential benefits of on-site stormwater management measures are difficult to quantify and capture. Over the long-run, we believe that qualitative costs and benefits of urban infrastructure elements will continue to be recognized and will significantly affect decisions on programs, policies, and investments.

R0022245

V. Approaches and Measures

You may apply a wide range of measures to your sites. As a reminder, not a limitation, consider the following potential strategies:

- *Tree plantings*. Tree foliage can hold and absorb or evaporate a significant portion of the rain falling annually on the diameter of the tree canopy.
- *Soil rehabilitation*. Aeration, soil amendments and other techniques can increase the infiltration rate of lawns. Certain grass species, by virtue of denser, deeper roots, can further improve infiltration.
- *Surface* infiltration basins. In some yards and many commercial landscapes, ponds, temporal "water gardens," and other basins can be designed to gather site runoff and hold/infiltrate it over varying periods of time.
- *Vegetated swales*. These features can infiltrate, attenuate, and treat runoff.
- *Disconnection of impervious surfaces*. Pitch drainage from driveways, sidewalks, and other pavements onto adjacent vegetated soil where it can infiltrate, not directly into street gutters. Divert low gutter flows similarly.
- *Street narrowing*. Common now in new developments, narrow streets calm traffic, increase green space, improve property values, and reduce impervious area.
- *Parking lot redesign*. Creative layout can incorporate "infiltration islands," filter strips, and other stormwater management features with no or little impact on the number of parking spaces.
- *Porous pavements*. Selectively apply porous concrete, porous asphalt, unit pavers (stone, brick, and concrete masonry), open-celled pavers, reinforced turf, gravel, organic mulch, decks, and grates to appropriate locations and uses.
- *Subsurface detention/infiltration chambers*. Made of gravel or manufactured components, varying depths and capacities of chambers can be installed under lawns and pavements to hold large volumes of site runoff during a storm and infiltrate that water to the subsoil in the following hours or days.
- *Roof leader disconnections*. Appropriate redirection of the leaders, re-grading of the landscape around a building, use of dry wells, and other techniques can infiltrate roof runoff without flooding basements.
- *Cisterns*. Some roof runoff can be captured in rain barrels or other cisterns and either used for yard and garden watering, car washing, etc., or released to dry wells or other infiltration systems once the storm passes.
- *Eco-roofs*. A modern variant on the sod roof, with lower weight and easier handling and maintenance, has been created and installed widely in Europe. Eco-roofs absorb water and evaporate it back to the air or grow incorporated plants, greening and cooling the building and cityscape.
- *Culvert daylighting*. Reopened stream corridors can include space in the cross-section for flood spreading and attenuation, permeable surfaces for infiltration, and diverse riparian plantings.
- *Site reconfigurations*. In the redevelopment context sites can be redesigned to reduce the quantity of pavement, or density can be increased to reduce the need for automotive transportation and the pavements it requires.
- *Interior water-use efficiency*. Many technologies to improve interior end-use efficiency are available that can, without sacrificing fixture or appliance performance, cut base sewer flows from buildings. Focus your efforts on exterior/landscape measures, but you may include interior recommendations in your write-ups if you wish.

R0022246

Required Charrette Products

Completion of certain work products is essential to the success of this charrette. The products specified below for the site teams will be incorporated into a final report to be distributed to area decision makers. The objective of this report is to inform and interest public officials, local professionals, and citizens in site-based stormwater management strategies. Our ultimate aim is to add to the local dialogue on wet weather management and urban redevelopment and rehabilitation, in order to improve choices on management strategies and produce multiple benefits.

The report will consist of a 14-16 page "high impact" color glossy summary, and a longer supplement. The summary will include the very best graphics from the charrette teams, short summaries of the site plans and selected elements, key policy recommendations, and a concise narrative tying together the charrette results. Its purpose is to pique interest and generate dialogue. The black and white supplement will include additional graphics, and more detailed descriptive text and policy findings. It will provide further information to enable communities to take the next steps in exploring and eventually implementing the strategies this charrette will outline.

Required Products: Site Teams

Site Plan Color Rendering

Each team must produce an illustrative color site plan at the scale of the base map provided. This plan will likely be reproduced in the color summary document. Provide a simple north arrow and simple reducible bar scale. Do not title the plan; a title will be provided in the final report.

Additional Graphics

As appropriate, elaborate on your plan and its elements with additional drawings and renderings. We ask you to produce at least 6 additional graphics; we hope you will be able to produce more. These should include a mix of detail plans, elevations, cross sections, and perspective sketches, and may include bird's eye views or other formats. Indicate materials and construction technique where this is considered significant. Choose scales as appropriate, keeping in mind any graphics may be considerably reduced in the final report. These graphics may be published in either the color summary or the black and white supplement, so choose ink line and color techniques appropriately. Obviously, focus most of your graphics on elements that have significant water quantity, water quality, or related urban community implications. Emphasize production of graphics that will communicate the look and feel of your recommended strategies to officials and citizens who are not design professionals and may not be familiar with interpreting technical drawings. Each additional graphic should be accompanied by a written description of approximately 100 words in addition to your overall project description.

Explanatory Narrative

Each team should develop a narrative of 800-1500 words describing your overall plan and recommendations. Explain the main strategies employed and briefly touch on key elements. Describe the water quantity and quality performance and the implications for the urban environment and urban quality of life. Indicate your suggestions for phasing of implementation over time and cost-sharing of measures between wet weather functions and other urban functions and agendas. Also identify any site-specific constraints to meeting the hydraulic performance or budget guidelines.

R0022247

Matrix of Benefits and Tradeoffs

Develop a simple table or matrix that concisely outlines (quantitatively or qualitatively) the benefits, and the costs or tradeoffs, of your retrofit recommendations across a range of functions and values: e.g., runoff quantity management, water quality, water supply, green waste management, mobility and access, energy use, air quality, employment, habitat protection and creation, recreation and leisure, beautification, economic and social development. You may delete, modify, and add to these categories as applicable to your site and recommendations. We suggest that you beginning recording potential benefits and tradeoffs early in your discussions to facilitate production of this matrix.

Outline/List of Implementation Issues and Suggestions

As in most charrettes, your designs and descriptions will be largely conceptual. We encourage you to develop as much detail as time allows. Aim to enable others to take up your ideas and run with them. Please keep and provide a list identifying: a) missing data and information, and design issues, that must be addressed to bring your ideas to fruition, b) known policy barriers to implementation of your designs, c) any suggestions you have for project finance, management, phasing, construction, etc., and d) key maintenance requirements for your designs. This document will be reproduced in the report supplement for the use of those who wish to pursue your concepts further.

Required Products: Policy Team

Products required of the policy team are described in the policy team brief later in this report. We expect the policy team to present many of its findings and recommendations as narratives, lists, and other text; however, we encourage the policy team to consider creative ways to communicate its results visually, with diagrams, flow charts, tables, graphs, drawings, or other means.

Additional Considerations

We prefer you minimize text added to plans, elevations, and other drawings. Err on the side of avoiding clutter. Keep in mind that any drawings may be substantially reduced in size in the final report, and text can be added to graphics later by the report editors if needed.

Each team will be provided a computer, disks, and access to a printer. Provide written materials in hard copy and on floppy disk. Electronic files should be in Word 5.1 or higher for Macintosh. You may use tables, bullets, different fonts and font sizes, the drawing function, and so on, but avoid stylistic choices that will lose content or format if changed to other styles for the final report (e.g. table or drawing formatting highly dependent on specific fonts or font sizes).

R0022248

Edgewood Transit Crossings

Location

This site lies at the center of Edgewood, a predominantly white collar community. A portion of an elevated railway bed running along the west side of the site will soon be developed as a busway by the Port Authority of Allegheny County (there is also some community interest in light rail here). Freight trains will continue to use the remaining railroad right-of-way. Swissvale Avenue is cut off by the railway bed, but nonetheless serves as an important local arterial that runs into the site from the northeast and then shunts south along the rail line. The site is surrounded by homes, schools, and small businesses. The Edgewood town hall is adjacent on the east side of the tracks, and a community building with a public library, swimming pool, and bowling alley lies just across the west side of the tracks. The Edgewood Town Center shopping center is located a few blocks south.

Structures and Landscape

Physical diversity and neighborhood activity are key features of this site. The defined site includes the elevated railway and a historic train station building designed by Furness, plus a small park adjacent to the station. Continuing across Swissvale Avenue, the site takes in a small commercial building (once a community grocery; now an architect's office with other small businesses), a small one-story dental office building, a multi-story apartment building, a large house, and a Presbyterian church. The closely spaced buildings mostly date from the early to middle decades of this century, and range in condition from fair to excellent. The site has a high proportion of impervious surface, but also has some gravel parking areas and small turf areas around several of the buildings. Race Street crosses under the tracks, as does a pedestrian walkway. Separate sanitary and storm sewers run underneath the major streets of this site, as well as water lines.

Drainage and Watershed Issues

Impervious surfaces are directly connected to separated stormwater sewers via street drain inlets and roof leaders. Rainfall carried off by the conveyance infrastructure contributes to high peak flows in Nine Mile Run, and carries oils, debris and other urban pollutants. Ground water recharge and downstream base flow are reduced. While the surface and fill of the elevated railway reportedly infiltrates water rapidly, the proposed busway could increase imperviousness of the site still further.

The street intersection here is at an artificial topographic low point. During intense storms street runoff enters the intersection from all directions, especially from Maple Street where fast-moving runoff on the steep slope bypasses drainage inlets. During intense rain events runoff ponds up in the intersection to the extent that it enters the door of the architect's office (a rise of at least one foot), and probably also the antique store in the basement of the train station. The latest ponding incident was in late August.

The old Edgewood train station, with the railway bed to the left.

R0022249

Soils and Geology

Alluvial deposits of the Carmichaels Formation (an alluvial terrace deposited by the ancient Monongahela River) may be 10 feet or more over most of the site. The underlying Casselman Formation bedrock consists of alternating layers of shale, siltstone, and sandstone, with some red beds and thin limestone and coal. The natural Culleoka soil was a moderately deep, well-drained soil on uplands. The texture was loam to clay loam throughout the profile. Permeability and water-holding capacity were moderate. The natural Rainsboro soil was a deep, moderately well drained soil with a fragipan. The natural geology, soil and topography have been strongly modified by urban cut and fill and by artificial drainage. Urban fill has been used especially to build up the railroad bed.

Social Issues

Edgewood residents consider the two sides of the track to be a single community unit, from the town hall on the northeast, to the community building on the southwest. This is the public center of Edgewood. There is considerable pedestrian traffic through the railroad underpass. A common route is from Maple Street, through the underpass, to the community building. The sidewalks along Swissvale Avenue and Edgewood Avenue are well-used, in part as a route to the shopping center 1/4 mile to the south. Because of the nearby schools and the community facilities, many of the pedestrians are children.

Looking up Swissvale Avenue from the train station.

The development of the busway is a major issue affecting Edgewood. In response to community concerns, the Port Authority (the regional transit agency) has eliminated the historic train station as a busway stop, instead moving the stop to the shopping center, and proposed a high noise barrier wall. Some town and regional residents are promoting a light rail alternative. The busway proposal includes a "linear park," which is essentially a sidewalk.

Design Considerations

Please refer to the design guidelines earlier in this brief for your general instructions. Among the possibilities for this site, we suggest you consider:

- Porous pavements for sidewalks, streets, driveways, parking lots.
- Trees along the busway for interception of precipitation, moderation of microclimate, moderation of bus emissions, and neighborhood beautification.
- Using subsurface of streets, sidewalks, driveways, & parking lots to detain or infiltrate runoff.
- Diverting some runoff from impervious surfaces into the busway for treatment in vegetated swales and subsurface infiltration and detention basins.
- Enhancing convenient and safe pedestrian access within and through the site, including concerns of universal accessibility.
- Enhancing the community focus of the locale by programming the reuse of the historic train station as a service-oriented public or commercial facility.
- Please give special attention to varied possibilities for disconnecting roof leaders from the conveyance infrastructure—this is likely to be an important technique for sewer and storm line flow reduction throughout the region.

R0022250

Hunter Park

Location

This site is in the upper reaches of the watershed, in a small valley surrounded by housing on slopes and hilltops. The Penn Avenue commercial district of Wilkinsburg lies approximately six blocks south. A number of neighborhood streets circle the park. Swissvale Avenue provides a straight shot up to the site; however, it changes from a major arterial at Penn Avenue to a neighborhood street and dead ends along the east side of the park. The former East Hills shopping center, scheduled for mixed use redevelopment, is located further upslope, but vehicle access to this area from the site is currently difficult, and pedestrian access is not provided. The boundary between Pittsburgh and Wilkinsburg lies at the upper (northern) reach of the park.

Structures and Landscape

Hunter Park is terraced up its headwater valley in four levels. The lowest level includes a small grassy area and a water play pool and fountain with a dolphin sculpture. This feature is relatively new but the water valve is broken and the water turned off by the city. Slightly higher is a larger open space and a basketball court in moderate to poor condition. The next level includes a baseball field and a set of tennis courts. These courts are in disrepair and are now used for municipal yard waste composting. The highest level consists of some smaller terraces and access points from the roads surrounding the park.

The site includes several row houses at the lower side of the park. These houses, and many in the surrounding neighborhood, are in very poor condition. Hunter Street extends part way up into the park. Its surface is asphalt overlaid over old masonry pavement. There is a vacant lot at the lowest end of the park, the corner of Hunter Street and James Street. Sidewalks in the area are in poor condition, and do not extend up to the park as one might expect.

Drainage and Watershed Issues

The park itself has a high proportion of pervious surface, mainly turf. The surrounding neighborhood is very dense—streets, sidewalks, and houses comprise most of the space. These impervious surfaces are directly connected to storm sewers via downspouts and drainage inlets. They contribute to stormwater pulses, sewer overflows, reduced base flow, and water pollution in the watershed below.

The stream through the park and downstream residential areas is culverted, as is all tributary drainage. A spring still exists in the slope above the highest park terrace. The lower density residential area above the park drains to the culvert running through the park. There is something of a swale currently around upper edge of the lowest level of the park. This drains to an inlet to the stormwater culvert, which then runs from the park across the James Street alley and between the residences below. This culvert backs up from time to time and the inlet grates are covered with debris.

The lowest terrace of Hunter Park, and surrounding houses.

R0022251

Soils and Geology

Bedrock here is the Casselman Formation—alternating layers of shale, siltstone, and sandstone, with some red beds and thin limestone and coal. These layers are locally prone to landsliding on steep slopes, particularly at seeps and springs and where natural slopes are steepened by cut or fill. Soils here lack the underlying alluvium of the other sites. The natural Culleoka soil was a moderately deep, well-drained soil on uplands. The texture was loam to clay loam throughout the profile. Permeability and water-holding capacity were moderate. Bedrock was 20 to 40 inches deep. A water table was not present in the soil. The natural geology, soil and topography have been strongly modified by urban cut and fill and by artificial drainage.

Social Issues

Wilkinsburg is a predominantly African-American community. Hunter Park is located in a less affluent area where housing conditions are often sub-standard, access to public transport is several blocks away, and there are practically no shops in the immediate vicinity.

All park facilities—baseball, basketball and dolphin fountain—are heavily used in season. The users come from residences mainly within a four-block radius. Many children congregated at the water feature when it was functional. Hunter Park needs more access from the upper level down. Except for one or two unofficial trails, the only access to the park is from below. The borough of Wilkinsburg has identified this park for potential redevelopment, in the hopes of creating a better amenity for a larger area, and of spurring revitalization of the neighborhood.

Design Considerations

Please refer to the design guidelines earlier in this brief for your general instructions.

Looking down on the upper terrace.

Among the possibilities for this site, we suggest you consider:

- Reinforcing a sense of continuity between the neighborhood and the park, through convenient pedestrian circulation and visibility.
- Rehabilitating all sidewalk pavements and the general streetscape.
- Porous pavements for rehabilitated sidewalks, park roads and walks, rehabilitated basketball courts, etc.
- Restoring permeable masonry street pavements by removing the asphalt layer. Rehabilitate the masonry surface, where necessary, by releveling and resetting in sand.
- Increasing street and park tree cover for interception of precipitation, moderation of microclimate, and neighborhood beautification.
- Providing additional parking in Hunter Park without increasing runoff.
- Rehabilitating and possibly realigning the basketball courts and playing fields. Reprograming the area now covered by the tennis courts/compost facility.
- Routing a daylighted stream segment or swales around the playing field edges.
- Diverting some runoff from surrounding houses and impervious surfaces into the park for treatment in vegetated swales.
- Celebrating the hillside spring in some manner.
- Creating or accommodating future links to areas below (Penn Avenue commercial area) and above (East Hills shopping center site).
- Please give special attention to ways of enhancing the economic and social fabric of the neighborhood.

R0022252

Regent Square Gateway

Location

This site sits at an important juncture—the boundary between Edgewood, Swissvale and Pittsburgh. It is also the site of the outfall of the Wilkinsburg Culvert, marking the transition from the developed upper watershed to the open portion of Nine Mile Run through and beyond Frick Park. Braddock Avenue, a key north/south arterial, runs by the site, and an interchange of the Penn Lincoln Parkway is adjacent. The Regent Square commercial district is located just uphill, and the Edgewood Town Center, a sub-regional shopping center, lies across the Parkway from the site.

Structures and Landscape

The site encompasses the building and parking areas of the former Foodland supermarket, plus the public right-of-way of old Braddock Road, and associated vacant space adjacent to the Parkway and around the culvert outfall and into adjacent Frick Park. The supermarket space on the main floor of the building is now vacant. A few offices in the lower floors are occupied. Potential uses which have been publicly discussed for the building include a used car dealership and a drug store. A terraced parking lot extends from the main level, supported by a retaining wall that joins to the east facade of the building, and cut into a steep hillside to the west. The remnant of old Braddock Road, cut off when the Parkway was built in the early 1950s, shows the cobble street and trolley tracks of the old urban pattern, before automobile dependence became so strong.

Looking down old Braddock Road at the former Foodland.

Nine Mile Run was culverted through the property in the 1930s. The Pennsylvania Department of Transportation later redesigned the infrastructure to accommodate the Parkway. Gas, water, and sewer lines run through the site, approximately along the grass edge of old Braddock Road. There is also a Bell Atlantic phone manhole just above the culvert outlet.

Drainage and Watershed Issues

This is the point where all of upper Nine Mile Run's stormwater culverts discharge to the surface. Flow in the Nine Mile Run channel here is a reflection of everything good and bad in the watershed: booming flood pulses, low base flow, poor water quality from urban pollutants and combined sewer overflows. The first several hundred feet of the channel is artificially straight, simple, impervious, and rigid, and contains little habitat structure.

There is considerable direct runoff through the site from the Braddock Avenue direction, reportedly flowing several inches deep during intense storms. The sewer manhole at Charleston and Braddock (one block up the hill) has been know to blow off during heavy rain events. Sheet runoff down old Braddock Road is actively eroding the slope where it falls into the Nine Mile Run channel, and contributes trash and other pollutants to the stream. The archway of the culvert is also in bad shape. The last 70 feet or so of the culvert appears to be constructed around the bridge over which Old Braddock Road used to cross the stream.

R0022253

Soils and Geology

The Glenshaw Formation bedrock here—alternating layers of shale, sandstone, siltstone, limestone, claystone, and coal—is overlain by unconsolidated alluvium deposits consisting of loam, silt, sand, gravel and clay in varying proportions deposited by Nine Mile Run before urbanization. Bedrock porosity and permeability vary locally with rock fracturing. Drainage through the alluvium is excellent above the water table. The natural Culleoka soil was a moderately deep, well-drained soil on uplands. The texture was loam to clay loam throughout the profile. The natural geology, soil and topography have been strongly modified by urban cut and fill and by artificial drainage. Fill for road and highway construction is significant in the original stream valley, overlying both the Glenshaw Formation and the stream alluvium. In depth, the fill extends approximately from the stream bed to the present-day ground surface. Laterally, the fill is confined to the original stream valley and does not extend under adjacent buildings or slopes.

Social Issues

Prior to the Parkway and constriction of the open stream channel, this area was an entrance to Frick Park. A chain link fence now prevents access to the trail leading to the park. A blazed path around the fence reveals that bicyclists, runners and walkers bypass the fence and trek to the park anyway. Erosion is cutting back the slope on which the unofficial trail passes.

Local municipalities are interested in redeveloping a safe entrance to the park here. This would be the functional entrance to the Nine Mile Run greenway through Frick Park, which will carry bike and foot traffic, and may generate its own demand for trail head parking. This would provide an important and highly visible entrance to the park for the Regent Square and Swissvale communities, as well as commuters entering the highway. It would also be the symbolic entrance to the greenway and the free-flowing portion of Nine Mile Run.

This area is now an unfriendly place for pedestrians. The sidewalk in the block immediately uphill of the site, coming down to the site from Regent Square, is encroached on by vegetation, and the street is entirely given over to cars.

The Jewish 'eruv' line runs alongside the right-of-way beneath the telephone lines on the Edgewood side of old Braddock Road. The eruv is a traditional Jewish means of marking the neighborhood. The eruv marks the boundary of the Squirrel Hill neighborhood for orthodox Jews.

Design Considerations

Please refer to the design guidelines earlier in this brief for your general instructions. Among the possibilities for this site, we suggest you consider:

- Porous pavements for sidewalks, streets, driveways, parking lots.
- Trees for interception of precipitation, moderation of microclimate, and moderation of vehicular emissions.
- Maintaining or restoring permeable masonry street and sidewalk pavements.
- Public or commercial, watershed-oriented or service-oriented programming for reuse of the building.
- Enhancing the convenience of pedestrian accessibility from surrounding neighborhoods.
- Using the subsurface of street, sidewalks, driveways, & parking areas to detain or infiltrate runoff.

R0022254

- Resolving sheet flow down old Braddock Road, and erosion around the culvert outfall.
- Creating foot, bicycle, and restricted vehicular access to Frick Park.
- Symbolically enhancing the discharge of Nine Mile Run's culverted flow into the open channel. Prominent visibility would be justified. It may be possible to daylight the channel further into the present culvert so that it pools where it currently leaves the culvert.

The site from the Penn Lincoln Parkway.

R0022255

Sterrett School

Location

Bounded by Edgerton Avenue, Reynolds Street, and Lang Avenue, this site is located in the South Point Breeze neighborhood of Pittsburgh, a middle-class, predominantly residential area. Traffic on the surrounding streets is light. A small market and an auto repair shop are located across Reynolds. Entrances to the upper Fern Hollow portion of Frick Park and to the Homewood Cemetery are nearby, as is the Frick Museum. Penn Avenue, a major arterial and commercial street, runs east/west roughly two blocks north of the site.

Structures and Landscape

Sterrett School is a three-story middle school built in the early part of this century. Grounds around the west and north sides are landscaped with turf, trees, and hedges. A new parking lot is located on the south side of the building, and a large paved play area lies adjacent to the east facade. East of this pavement, and a few feet lower, a ball field stretches across the middle portion of the block. The east-most portion of the block encompasses 7 homes (two are duplex units) and their associated yards and garages. The ages of these structure range from approximately 30 to 100 years. A short alley runs between these houses.

Water lines and combined sewer lines run along the center or far side of surrounding streets. Gas lines run along the near side of Reynolds Street. A 36" combined sewer line cuts diagonally under this block from the Reynolds/Le Roi Road intersection southeast toward Frick Park.

Drainage and Watershed Issues

The main stream of the Fern Hollow headwater once ran through the site where the low alley between the houses now lies. It is now culverted, along with all tributary drainages, in combined sewer lines.

Impervious cover is high for much of this site: roofs, streets, sidewalks, driveways, and parking lots. Runoff from these surfaces is directly connected to combined sewer lines via street drain inlets and roof gutter leaders, contributing to downstream CSOs. Diversion of rain water across impervious surfaces and to combined sewers also reduces ground water recharge and downstream base flow.

The Sterrett School field is a notable exception to the high impervious cover in the area. Other existing exceptions are the yards of residences and the school. The school field surface is compacted and drains toward the residences. Runoff from the field flows into the low area between residences, and contributes to flooding of the alley and some of the house basements. Drain pipes in the playing field and a wood barrier along the low side of the field have not proved effective.

Playing fields and some of the houses on the east side of the block.

R0022256

Soils and Geology

Bedrock here is the Casselman Formation—alternating layers of shale, siltstone, and sandstone, with some red beds and thin limestone and coal. The bedrock is at or near the surface at the uppermost corner of the site, but is overlain in the downhill parts of the site by an increasing depth of alluvial terrace deposits from the Monongahela River's pre-glacial channel. These unconsolidated deposits consist of interstratified sand and gravel, boulders, silt and clay, and are easily excavated and well-drained above the water table. The alluvium is shallow at the upper (southwest) corner of the site and gets progressively deeper down-slope towards the east end of the site. The natural Guernsey soil was a deep, moderately well drained soil on uplands. The texture was silt loam to clay and silty clay throughout the profile. Permeability and water-holding capacity were moderate. The natural soil has been strongly modified by urban cut and fill and by artificial drainage.

Sterrett School from across Reynolds Street.

Social Issues

Pedestrian use of sidewalks is generally active, safe and convenient throughout the neighborhood. The school's field is used as a neighborhood resource, with games scheduled frequently throughout the week. The small exercise equipment area towards the southeast corner of the playing field is soon to be replaced with a "tot lot" area, in exchange for the loss of a tot lot previously located in the area recently paved for additional parking on the south side of the building. Staff and students have been replanting the west and north grounds over the years.

Design Considerations

Please refer to the design guidelines earlier in this brief for your general instructions. Among the possibilities for this site, we suggest you consider:

- Realigning, re-grading, and re-landscaping the field sports facilities, school yards and surroundings for infiltration and other functionality improvements.
- Porous pavements for streets, sidewalks, driveways, and parking areas. Streets may have an old permeable masonry pavement under the asphalt overlay; if so the masonry could be uncovered by milling off the asphalt.
- Street narrowing.
- Diverting some runoff from the school's impervious surfaces into the field area for surface or subsurface detention, infiltration and treatment.
- Reconfiguring the gutter below the playing court.
- Rehabilitating the playing field surface to loosen soil, improve infiltration, reduce runoff, and combat further compaction.
- Reconfiguring some components of the residences to escape flooding and prevent direct contribution of on-site runoff to storm sewer flow.
- Please give special attention to varied possibilities for disconnecting roof leaders (at the school and the residences) from the conveyance infrastructure—this is likely to be an important technique for sewer and storm line flow reduction throughout the region.

R0022257

Policy Team

Developing Policy Options for Nine Mile Run: Integrating Infrastructure, Ecology, and Urban Agendas

This section of the brief outlines an approach and specific tasks for the policy team. We are asking you to outline an integrated policy framework, evaluate existing policies, and suggest institutional forms and programmatic linkages. You are asked to consider municipal, county, state, and perhaps federal policies and institutions. Your recommendations on policies and institutional structures should resonate in the particular context of the Nine Mile Run Watershed and apply well beyond. Your work will necessarily be broad in scope, but you should pay special attention to enabling implementation of the site-scale retrofit techniques under consideration by the design teams. We expect that the designers will note policy barriers and opportunities as their work proceeds, so we have scheduled a plenary session on Friday afternoon for sharing of ideas between the design teams and the policy team.

Below we propose a philosophical approach and general goals for integrated urban watershed management, and then lay out the policy and institutional topics we would like you to address.

Principles

Principle 1: Urban stormwater policy and its attendant retrofits involve different issues, opportunities and constraints than greenfield development policy.

Retrofitting century old urban stormwater/sewer systems requires creativity and strategic intervention to achieve the long-term goals of water quality as described in the Federal Clean Water Act. Policy crafted for use in the urban context should encourage innovation and experimentation within specific standards for monitoring and institutional programs for long-term efficacy analyses. Existing ordinances and other policies should be analyzed for potential constraints on urban innovation.

Principle 2: Policies and institutions should enable effective feedback between receiving water conditions and actions in the contributing watershed.

Urban watersheds have traditionally been managed as infrastructure systems, ignoring the underlying ecosystems which are often displaced and always affected. Monitoring is often a regulatory agency reaction to suspected problems rather than an integrated tool of infrastructure management and maintenance. This process has been inefficient and is perceived to be politically motivated (external) rather than a responsibility for local governments (internal) to find the best means of managing infrastructure systems. Better ways of achieving feedback between ecosystem conditions in receiving waters and effective upstream infrastructure management are sorely needed.

Principle 3: Policies should promote the management of storm sewer systems as infrastructure extensions of the natural ecosystem and its related phenomena.

Hydrologic and ecological processes do not just occur downstream. Infiltration and bio-filtration are elements of natural ecosystem function which can be used to manage wet weather at each property and site in a watershed. Policies for infrastructure, and institutions should enable the use of the natural capacity of soils, vegetation, and microorganisms to regulate water flows and remediate pollutants. Society must identify and remove barriers to using the remediative capacities that exist in and under urban landscapes, streets, and buildings.

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Principle 4: Urban stormwater policies should communicate the ideas of limits, citizen participation and program coordination as part of the agenda of urban retrofit.

Stormwater has been viewed as an ever-expanding byproduct of urban development and growth. Expecting century old systems with a history of narrowly-focused retrofits to meet contemporary regulations and societal needs mixes best intentions with difficult realities. Policies should encourage local understanding of the limits of existing systems and foster implementation of alternatives which can mitigate source flow. Wet weather management does not occur in a vacuum. The continual process of building and rebuilding the cityscape includes multiple other functions and agendas: housing, transportation, economic development, beautification, recreation, social justice, energy management, solid waste management, and much more. Resolving a history of hydraulic expansion, poor maintenance and little "big picture" oversight is a long-term goal which must be integrated into other programs and efforts to improve the physical, economic, and social elements of urban environments. This coordination can and should include citizen participation, cost-sharing, innovative funding strategies and potentially, new institutions. If we see stormwater as an opportunity rather than a problem, funding sources for coordinated infrastructure intervention may be found.

Goals for Integrated Urban Watershed Management

Successful policies and institutions are motivated by clear yet ambitious goals. We suggest the following baseline goals for policies and institutions directed towards integrated urban watershed management:

1. Equitably eliminate existing water quality and water quantity problems.
2. Restore and steward the ecosystem functions in the watershed.
3. Manage the infrastructure and ecosystem to maximize benefits and minimize costs.
4. Enable alternatives and experimentation in the pursuit of the above.
5. See infiltration and bioremediation as a cost-effective watershed management method, process and resource.
6. Enable the potential evaluation and rehabilitation of any site in the watershed.
7. Create synergies between water quantity and quality objectives and accomplishment of other urban agendas.

Tasks for the Policy Team

Take as the audience for your findings and recommendations the decision makers and public officials who may be asked to buy into a new approach and develop, approve, implement, and enforce its specifics. Make your work clear and meaningful to this audience. Tasks I and II are general and preparatory; tasks III and IV should be the meat of your work.

I. Elaborate a Philosophy

Spend an early and small portion of your time considering the principles mentioned above. Please elaborate briefly on these principles and suggest any others you feel are fundamental to establishing a holistic approach to urban watershed wet weather management (in particular, an approach that incorporates site-based multi-functional retrofit opportunities).

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II. Construct a Comprehensive Policy Framework

Policies and institutions are typically developed piecemeal, as responses to very particular needs and the crises of the day. The result is often a mishmash of laws, codes, regulations, departments, special districts, and so on that may not work together well and often conflict. Rarely do we step back and look at how the various issues we are trying to address may be related.

The Pittsburgh region currently faces a sanitary sewer overflow "crisis." Regulatory pressures on local communities to eliminate SSOs are rapidly mounting. At the same time, ALCOSAN and local authorities are planning combined sewer overflow reduction strategies. And local streams such as Nine Mile Run exhibit significant damage from stormwater runoff. At this juncture, we suggest it is imperative to step back and examine how we can be sure that:

- policies and institutions to address these and other problems do not conflict,
- potential solutions are not disallowed by existing or new regulations, and
- approaches to these issues are sufficiently comprehensive and efficiently integrated.

The current status of Nine Mile Run in Frick Park.

This charrette's policy team can advance the regional dialog on wet weather management by outlining a comprehensive framework for evaluating policies and institutions relative to these three standards. We ask you to do the following:

A) Identify *goals* that any code, legislation, program, management authority, or other policy or institution could potentially adopt, and at a minimum should not oppose. These may include the general goals mentioned above and other general and specific objectives.

B) Identify *functions* that must be achieved for successful, comprehensive watershed management. Broadly, these could include review and approval of development and retrofit proposals, coordination, funding, construction, monitoring, evaluation, education and promotion, maintenance, and much more. Please elaborate.

C) Identify *criteria* for evaluating policies and institutions. For instance, these could include political acceptability, stakeholder involvement, ease of implementation, enforceability, flexibility, ability to charge user fees, ability to attract inter-governmental financial transfers, and much more. Please elaborate.

As you address these tasks, pay particular attention to the goals, functions, and criteria relevant to enabling implementation of site-based retrofit technologies for infiltrating, bioretaining, detaining, or treating stormwater runoff.

III. Evaluate Policies and Institutions Against the Comprehensive Framework

Please evaluate a range of existing and potential policies and institutions against the goals, functions, and criteria you outline, and:

A) Identify where there are conflicts with an integrated urban watershed approach, missed opportunities, or inefficiencies in application or integration.

B) Suggest positive changes to reduce conflicts and improve integration.

A number of documents presenting important policies and institutions will be available to the team, including:

- Codes and regulations: building codes, plumbing codes, drainage regulations, sewer regulations, zoning ordinances, road design standards, etc.
- Laws and policies governing water quality, health administration, stormwater management, sewer facilities, etc.

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- Proposals for new institutional forms such as watershed authorities, special sewer or stormwater districts, etc.

You will not have time to review all these documents, so the team must determine which of these items are most important to evaluate. Pay particular attention to provisions that could preclude or enable site-based retrofit opportunities.

IV. Suggest Linkages with Other Urban Agendas and Programs

As noted throughout this brief, site-based techniques to infiltrate, bioretain, detain, and treat stormwater runoff can produce many non-stormwater benefits. This suggests that implementation of these techniques could be spurred or assisted through linkages with programs aimed at economic development, traffic management, transit, recreation, habitat creation, solid waste management, employment, and more. Thinking through how to foster these linkages is one of your most significant tasks. At a minimum, please:

A) Outline likely areas of municipal and multi-institutional cooperation.
B) Suggest specific programmatic linkages that could be made.
C) Recommend communication targets and tools to increase awareness of the multi-benefit potential of site-based retrofit techniques.
D) Recommend specific actions to further outline, investigate, and develop collaborations between officials and citizens interested in sewer and stormwater management and those with other interests and agendas.

As you address linkages and programs, we also ask you to give some consideration to this potentially radical thought: in this day and age, and in the context of urban watersheds, it may be appropriate to de-emphasize highly detailed ordinances and standards for the built environment, and to instead focus more energy on general goals and programmatic approaches to their achievement. Ordinances and standards may be most appropriate for the blank slate of new development. In the city, each site comes burdened and enriched with history and elaborate context. Sustainable design and urban management requires that we be responsive to the particular constraints and opportunities of each site, and adapt our techniques in many ways that may be precluded by a too-strict regulatory approach. On the other hand, precise regulations are an antidote to incompetence and disregard of the effects of one's actions on others. How can we best enable adaptive design and free ourselves to expect the best from urban retrofits and redevelopment, without creating an opening for sloppiness and greed?

Required Products

We expect the policy team to present many of its findings and recommendations as narratives, outlines, lists, and other text; however, we encourage you to consider creative ways to communicate your results visually, with diagrams, flow charts, tables, graphs, drawings, or other means.

Narrative Summary

Prepare an 800-1500 word summary of your findings and recommendations. This should concisely present your ideas on integrated urban watershed policies and institutions, and note some specific findings and recommendations that will be of greatest interest and import to local decision makers. This piece will likely be published in the summary document.

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Findings/Recommendations on Tasks I to IV

Develop documents to present your general and specific findings and recommendations on the tasks indicated above. You may use whatever approaches—narratives, outlines, lists, matrices, spreadsheets, graphics—you determine to most efficiently capture and convey your work. These materials will most likely be published in the supplementary document, but may be excerpted for the summary.

Your team must cover a lot of ground. You may wish to split the team into subgroups to address some of the tasks given above. Please keep in mind the available time, and do not get bogged down in any one area. Where you have disagreements, note these and move on. Your job is to identify issues and outline approaches and potential solutions. Your recommendations will be suggestive. Others can take up and refine your ideas.

"The people have a right to clean air, pure water, and to the preservation of the natural, scenic, historic, and esthetic values of the environment. Pennsylvania's public natural resources are the common property of all the people, including generations to come. As trustees of these resources, the Commonwealth shall conserve and maintain them for the benefit of all the people."

—Constitution of the Commonwealth of Pennsylvania, Article I, Section 27.

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Participant Biographies

Hugh Archer, Ph.D., P.E., DEE, is Deputy Secretary for Water Management, Pennsylvania Department of Environmental Protection. He has extensive experience in the areas of water quality, water resources, drinking water, sewage facilities, stormwater, dam safety, flood control projects and wetlands. Dr. Archer has also served as the regional water quality manager in the Southwest Regional Office, where he gained much experience dealing with permits for wastewater discharges, sewage planning, and grant administration. *Pennsylvania Dept of Environmental Protection, P.O. Box 2063, Harrisburg, PA 17105-2063. (717) 772-5996.*

Lucia Athens is Senior Resource Conservation Planner for Seattle Public Utilities, where she is responsible for sustainable design and construction programs for the city of Seattle. She holds degrees in landscape architecture and architectural studies, and has taught design in the University of Georgia School of Environmental Design and the University of Washington Department of Landscape Architecture. She helped develop the Green Builder Program of the city of Austin, Texas, and has consulted on many site and community design projects. *Seattle Public Utilities, Community Services Division, 710 Second Avenue, Seattle, WA 98104. (206) 684-4643.*

Bob Bingham is an Associate Professor of Art at Carnegie Mellon University. He is also a Research Fellow in the STUDIO for Creative Inquiry and a Co-Director of the Nine Mile Run Greenway Project. Mr. Bingham's current research focuses on the landscape within the cityscape environment. His work explores the creation of site-specific installations for public places such as the urban built environment. His research also includes developing strategies for a dialogue between personal and public identity and beautifying public spaces for aesthetic inquiry. *STUDIO for Creative Inquiry, Carnegie Mellon University, Pittsburgh, PA 15213-3890. (412) 268-3673.*

Fred Bonci is a founder of the firm LaQuatra Bonci Associates, which provides landscape architectural and planning services for residential, commercial, institutional, and environmental projects. He has experience in urban design, especially the planning and design of urban neighborhoods, community planning, and public open space projects. Mr. Bonci is currently leading the Pittsburgh Strategic Parks Initiative. He received his Bachelor of Science in Landscape Architecture from Pennsylvania State University in 1973. *LaQuatra Bonci Associates, 95 South 10th St., Pittsburgh, PA 15203. (412) 488-8822.*

Bill Browning is the founder of Rocky Mountain Institute's Green Development Services. He has a Master's degree in real estate development from the Massachusetts Institute of Technology, and received MIT's Charles H. Spaulding Award for distinguished alumni in the real estate profession. His Green Development consulting projects include new towns, many building renovations, Wal-Mart's Eco-Mart, the Sydney 2000 Olympic Village, Monsanto's corporate headquarters, Greening the White House, and the Pentagon Renovation. *Rocky Mountain Institute, 1739 Snowmass Creek Road, Snowmass, CO 81654. (970) 927-3807.*

Thomas Cahill is a Professional Engineer and President of Cahill Associates, a firm specializing in water resources management, environmental planning, and sustainable site development. He has conducted or directed numerous watershed management studies, including development of three dimensional models of ground water movement and pollutant migra-

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tion, and ground water base flow models. Cahill Associates is a national leader in the application of porous pavements, underground storage and infiltration beds, and other innovative stormwater management systems. *Cahill Associates, 104 South High Street, West Chester, PA 19132. (610) 696-4150.*

AB Carl is a Wilkinsburg resident who has recently retired from the Allegheny County Planning Department with twenty years of experience. He currently is the Economic Development Chair for the Wilkinsburg Economic Development Group – East. Mr. Carl is also treasurer for the Turtle Creek Watershed Authority and chair of Penn's Corner, a Resource Conservation and Development organization for the nine county area of Southwestern Pennsylvania. *901 William Penn Court, Pittsburgh, PA 15221 (412) 371-1614.*

Tim Collins is a Research Fellow in the STUDIO for Creative Inquiry, and Co-Director of the Nine Mile Run Greenway Project. Mr. Collins' creative practice and research interests are based in aquatic infrastructure (utilitarian and ecological) and the attendant cultural responses. Natural systems and the notion of ecologically defined community inform his approach. He has explored and created work and studies for a variety of waterfronts (with co-director Reiko Goto and others). *STUDIO for Creative Inquiry, Carnegie Mellon University, Pittsburgh, PA 15213-3890. (412) 268-3673.*

Patrick Condon holds the James Taylor Chair in Landscapes and Liveable Environments in the Landscape Architecture Department of the University of British Columbia. In that capacity he has organized a series of international design charrettes for urban sustainability. He also facilitated the 1997 Second Nature charrette for urban site retrofits in Los Angeles. From 1981 to 1983 he was the Director of Community Development for the City of Westfield, Massachusetts and from 1984-1991 he taught landscape architecture at the University of Minnesota. He is a partner in the design and planning firm of Moriarty/Condon Ltd. *Moriarty/Condon Landscape Architects & Planners Ltd., 1661 West 2nd Ave., Suite 102, Vancouver, British Columbia V6J 1H3. (604) 730-6987.*

Bruce Ferguson is a Professor of Landscape Architecture and Director of the Master of Landscape Architecture program at the University of Georgia. He is the author of *Stormwater Infiltration*, the standard professional reference in its field, *Introduction to Stormwater*, and 130 scientific and professional papers on environmental management of urban watersheds. He has participated in the setting of urban design guidelines to protect runoff quality through the International Life Science Institute's stream restoration program in Atlanta, the Second Nature charrette in Los Angeles, the Start at the Source manual for San Francisco, and additional projects in Florida, Georgia and New York. Mr. Ferguson is a recipient of the Council of Educators in Landscape Architecture's Outstanding Educator Award, the highest career award for landscape architectural education in North America. He is a Pittsburgh native who received the MLA degree at the University of Pennsylvania and practiced in the Allegheny County region for several years before commencing his academic career. *School of Environmental Design, University of Georgia, Caldwell Hall, Athens, GA 30602-1845. (706) 542-4720.*

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Rebecca Flora is the Executive Director of the Green Building Alliance where she works to integrate the use of 'green' technology and development practices. She received her Masters in Urban and Regional Planning from Virginia Polytechnic Institute & State University. From 1991 to 1997, she worked as the executive director for the South Side Local Development Company, where she managed a development plan that revitalized the Pittsburgh South Side's economy and preserved historic characteristics. She is also a part-time faculty member at the H. John Heinz III School of Public Policy and Management at Carnegie Mellon University. *Green Building Alliance, 64 South 14th St., Pittsburgh, PA 15203. (412) 431-0709.*

Mike Foreman is Local Government Policy Specialist with Pennsylvania's Center for Local Government Services in the southwest regional office. Mr. Foreman provides local government officials with policy guidance and advice, technical and consultive assistance and training in the areas of general government management and administration, financial management, revenue and taxation matters, intergovernmental cooperation and economic and community development. *Governor's Center for Local Government Services, Southwest Regional Office, 1403A State Office Building, 300 Liberty Avenue, Pittsburgh, PA 15222. (412) 565-5199.*

David French is currently the Vice President of Sales at L. Robert Kimball & Associates, Inc. He formerly served as project manager and principal planner for water resource related issues for the Allegheny County Planning Department. Mr. French's duties included preparation of comprehensive hydrological plans for county watersheds, coordination of storm water management projects and plans, reviewing federal and state policy initiatives, and developing county and municipal stormwater funding approaches. *L. Robert Kimball & Associates, 415 Moon Clinton Rd., Coraopolis, PA 15108. (412) 262-5400.*

Kevin Garber is a shareholder in the Environmental, Health and Safety Group of Babst, Calland, Clements and Zomnir, P.C. He graduated with distinction from the Pennsylvania State University (B.S. in Biology). Mr. Garber took his M.S. in Oceanography and Limnology from the University of Wisconsin, his Ph.D. in Ecology from the University of Pittsburgh and his law degree from the Duquesne University School of Law. Mr. Garber is on the advisory board for the Three Rivers Wet Weather Demonstration Program. *Babst, Calland, Clements and Zomnir, P.C., Two Gateway Center, Pittsburgh, PA 15222. (412) 394-5400.*

Ray George is a Public Liaison for the U.S. Environmental Protection Agency, Region 3. 303 Methodist Building, 1060 Chaplane Street, Wheeling, WV 26003. (304) 234-1234. Caren Glotfelty is Maurice K. Goddard Professor of Forestry and Environmental Resource Conservation at Pennsylvania State University. Ms. Glotfelty has professional interests in environmental resources policy, especially in the areas of water management, land use, and sustainable development. For the past eighteen months she has served as co-chair of Governor Ridge's 21st Century Environment Commission. She also worked with the Interstate Council on Water Policy, which developed a National Water Policy Charter that measures and evaluates water policy proposals under consideration by Congress, state legislatures, and state and federal executive agencies. *School of Forestry, Pennsylvania State University, 101 Ferguson Bldg., University Park, PA 16802-4300 (814) 863-2506.*

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Reiko Goto is a Research Fellow in the STUDIO for Creative Inquiry and Co-Director of the Nine Mile Run Greenway Project. Ms. Goto's creative practice and research interests lead to natural systems and the notion of ecologically defined community. Her research and production has been principally public or "civic" in nature. Over the last year, she has developed an innovative brownfields curriculum for children, which merges inquiry-based learning with the contextual analysis of local post-industrial properties. *STUDIO for Creative Inquiry, Carnegie Mellon University, Pittsburgh, PA 15213-3890. (412) 268-3673.*

Fran Greene is a Ph.D. candidate in the Department of Civil and Environmental Engineering at Pennsylvania State University. Prior to this he worked with Momonee and Associates in Bryn Mawr, Pennsylvania. *Department of Civil & Environmental Engineering, Pennsylvania State University, 212 Sackett Bldg., University Park, PA 16802. (814) 863-7304.*

Sandra Heard is an architect with MacLachlin, Cornelius, & Filoni in Pittsburgh. She has worked as an architectural consultant, and an architect, specializing in site analysis, restoration, and renovation in New Orleans, Louisiana. She is a Peace Corps veteran in the Dominican Republic and received a Master of Theology from Xavier University. *1522 Garfield Ave., Pittsburgh, PA 15212. (412) 231-5790.*

Walter Hood is an Associate Professor in the Department of Landscape Architecture and the Urban Design program at the University of California, Berkeley. His firm, Hood Design, is committed to the reconstruction of urban landscapes in ways that reflect their place, time, and the social uses that make them useful and memorable to the people who use and reside within them. Recent projects have included parks, squares, streetscapes, schools, housing projects, and urban stream corridors. *Hood Design, 3016 Filbert Street, Studio #2, Oakland, CA 94609. (510) 595-0688.*

Michael Hough is a Professor of Environmental Studies at York University and Principal of the internationally known design and planning firm Hough, Woodland, Naylor, Dance, Leinster. He has authored dozens of articles on urban design, ecological processes, and environmental planning principles, and several books, including *Cities and Natural Process.* Integrative stormwater management is a frequent theme in his work, and his many projects have ranged in scale from site designs to regional landscape plans. *Hough, Woodland, Naylor, Dance, Leinster, 916 The East Mall, Suite B, Etobicoke, Ontario M9B 6K1. (416) 620-6577.*

Mike Hullihan is the Director of Engineering and Construction at the Pittsburgh Water and Sewer Authority. *441 Smithfield St., Pittsburgh, PA 15222. (412) 255-8935.*

Greg Hurst is Professional Engineer and Principal, with EDAW, Inc., an international natural resources management and development services firm, where he directs the Site Engineering Services Team. He has engineered domestic water systems, sanitary and storm sewers, irrigation systems, and water features for dozens of projects around the world. A notable recent project on the Red Sea used small, dispersed underground chambers to infiltrate water for a zero discharge development. Mr. Hurst is a past president of the Fort Collins, Colorado Storm Drainage Board. *EDAW, Inc., 240 East Mountain Avenue, Fort Collins, CO 80524. (970) 484-6073.*

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Alex Hutchinson, founding member of Hutchinson and Sons Engineering, has been the municipal engineer for Edgewood Borough for over thirty years. *Hutchison and Son Engineering, 508 Rebecca St., Pittsburgh, PA 15221. (412) 241-3000.*

Peggy Johnson is an Associate Professor with the Department of Civil and Environmental Engineering at Pennsylvania State University. Dr. Johnson's research interests include civil engineering, river hydraulics, and watershed modeling. She received her Ph.D. in Civil Engineering in 1990 from the University of Maryland, specializing in water resource engineering. Dr. Johnson also teaches both undergraduate and graduate courses in hydrosystems engineering. Dr. Johnson is the geomorphologist on the Nine Mile Run ecological assessment team. *(814) 865-1330.*

Alexis Karolides is a registered architect and a Senior Research Associate with Rocky Mountain Institute's Green Development Services. She has studied physics and researched solar energy and other alternative energy systems. Ms. Karolides has worked in commercial, institutional, and industrial architecture for the Texas Parks and Wildlife Department, the Texas Governor's Energy Office, and the architecture firm Susman, Tisdale, Gayle in Austin, Texas. Current consulting projects at RMI include Monsanto's corporate campus, Aspen Skiing Company projects, green spec homes for a Hines development, and the New Mexico Department of Energy building, among others. *Rocky Mountain Institute, 1739 Snowmass Creek Road, Snowmass, CO 81654. (970) 927-3807.*

C. Noel Kennard is a registered architect at Burt Hill Kosar Rittelman Associates in Pittsburgh. Recently relocated from Toronto, Mr. Kennard was the sole principle in ICOSA Project Development. As a consultant, ICOSA undertook hotel resort projects and low-income housing developments in Mexico and the Caribbean. Mr. Kennard has extensive experience working with developers on a local and international basis and with community groups in Toronto. *Burt Hill Kosar Rittelman Associates, 650 Smithfield Street, Suite 2600, Pittsburgh, PA 15222-3907. (412) 394-7000.*

Georgina King has a Bachelor of Landscape Architecture with honors from Lincoln University. She worked in New Zealand for 3 years, involved primarily in community based and residential design projects. She was awarded the John. R. Templin travelling scholarship for 1998 in landscape architecture, with a primary focus on waterway restoration and development. She interned at Carnegie Mellon University's STUDIO for Creative Inquiry from June to August, 1998 and is currently interning with Michael Hough Associates in Toronto, Canada. *Hough, Woodland, Naylor, Dance, Leinster, 916 East Mall, Suite B, Etobicoke, Ontario, Canada, M9B6K1, (416) 620-6577.*

Bob Kobet is the Director of Green Building Services, Conservation Consultants, Inc. Mr. Kobet's current work includes several green building and sustainable community design and development projects. He holds a Master of Science in Sustainable Systems from Slippery Rock University, a degree he was instrumental in creating. As a member of the Board of Directors of the Pittsburgh Chapter of the American Institute of Architects and owner of Energy Design Associates, Mr. Kobet has extensive experience in the field of green design, architectural consulting, and sustainable community development. *Conservation Consultants, Inc., 64 South 14th Street, Pittsburgh, PA 15203-1548. (412) 431-4449.*

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Mary Kostalos is an ecologist and Professor of Biology at Chatham College and Co-Founder of the Rachel Carson Institute. From 1992 to 1997, Ms. Kostalos was Director of the Environmental Studies Program at Chatham College. Dr. Kostalos first tested the waters of Nine Mile Run for her doctoral thesis in the late 1960s and has returned to the site with her students in the intervening years. Dr. Kostalos has been an advisor for the Nine Mile Run Greenway Project almost from its inception. *Chatham College, 7500 Kensington, Pittsburgh, PA 15221. (412) 365-1215.*

Petra Kuehl is currently earning a Masters in Landscape Architecture from the University of Guelph in Toronto, Ontario, Canada. Her thesis is a comparative analysis of different soils and microclimates in the Nine Mile Run valley to quantitatively describe the wide diversity of vegetative maturity present in the area. After receiving a Biology degree from the Free University of Berlin, Ms. Kuehl completed research in molecular biology in Berlin and plant genetics at McGill University in Montreal. Ms. Kuehl received a Masters in Public Health from the University of Pittsburgh. She also has found time for a few years to manage her own landscape design firm in Toronto. *79 Elwood Boulevard, Toronto, Ontario, Canada, M5N1G9. (416) 484-1511.*

Suzanne Lami is a principal of Lami Grubb Architects with 19 years of experience in architectural practice. She founded her firm in 1993 as a sole proprietorship; the firm has grown to staff eleven. She received her Bachelor of Architecture Degree in 1979 from Carnegie Mellon University. Ms. Lami's offices are located on the Edgewood Transit Crossings charrette site. *Lami Grubb Architects, 100 East Swissvale Ave., Pittsburgh, PA 15218. (412) 243-3430.*

Jack LaQuatra is a founder of the firm LaQuatra Bonci Associates, which provides landscape architectural and planning services for residential, commercial, institutional, and environmental projects. He has experience in site and land use planning and recreational planning. Mr. LaQuatra designed The Village at Washington's Landing on Herrs Island and its public rivertrail, and the new neighborhoods, including Summerset, on the Nine Mile Run development site in Pittsburgh. He received his Bachelor of Science in Landscape Architecture from West Virginia University in 1973. *LaQuatra Bonci Associates, 95 South 10th St., Pittsburgh, PA 15203. (412) 488-8822.*

Durla Lathia, P.E., is the Chief of the Stormwater Planning and Management Section of the Pennsylvania Department of Environmental Protection's Bureau of Watershed Conservation. *Rachel Carson State Office Building, PO Box 8555, Harrisburgh, PA 17105-8555. (717) 772-4048.*

Chris Leininger is an architect and owner of Sustainable Home Design, a company formed to pursue design projects that focus on sustainability and environmental design principles. Mr. Leininger has a certificate in Bau-Biologie and has provided seminars on Straw Bale Construction. He has consulted with Pittsburgh non-profits and architectural firms including Conservation Consultants, Inc. and Gardner + Pope Architects. *690 7th Street, Beaver, PA 15009. (724) 773-0703.*

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Paul Leonard is a Certified Fisheries Scientist and Senior Project Manager with EDAW, Inc., an international natural resources management and development services firm. His watershed planning experience includes watershed characterization and study design, GIS mapping and analysis, water quality and hydrologic analysis and modeling, stream habitat and biological assessment, non-point source pollution BMPs and control strategies, managing expert panels and consensus-building. He developed a watershed restoration master plan for a small Atlanta urban watershed, and is currently managing EDAW's involvement in the Big Creek Watershed Protection Plan in Atlanta, Georgia, and the Stormwater Master Plans for the Deep Creek and Marsh Creek watersheds in Fulton County, Georgia. *EDAW, Inc., 3475 Lenox Road, Suite 100, Atlanta, GA 30326. (404) 365-1110.*

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Suzanne Meyer is a registered landscape architect with professional experience in land development and environmental impact assessments in the private, public, and non-profit sectors. She recently produced "From Cape Cod to the Bay of Fundy: An Environmental Atlas of the Gulf of Maine" for the Island Institute in Rockland, Maine. With this recent experience in content development and project management, she returned to her hometown, Pittsburgh, Pennsylvania, to establish Image Earth. *Image Earth, 1604 Buena Vista St. #1, Pittsburgh, PA 15212. (412) 321-4648.*

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Christine Mondor is an architect at Gardner + Pope Architects, dealing with design of sustainable buildings and systems. She maintains a close tie to the landscape through her proj-

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Jan Oliver is CSO program manager with the Allegheny County Sanitary Authority (ALCOSAN). She helped developed, and is currently the Assistant Manager of the 3 Rivers Wet Weather Demonstration Program. *ALCOSAN, 3300 Preble Ave., Pittsburgh, PA 15233-1092. (412) 734-8351.*

Andy Otten is a Master of Arts candidate in Landscape Architecture in the College of Arts and Architecture at Pennsylvania State University. He is designing and managing the Spring Creek Riparian Restoration Project in Bellefonte, Pennsylvania. *c/o Pennsylvania State University, Landscape Architecture Department, Engineering Unit D, University Park, PA 16802-1429. (814) 865-7832.*

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Henry Prellwitz is a Ph.D. candidate in Geology at the University of Pittsburgh. Mr. Prellwitz's thesis topic is a geochemical and environmental study of the Nine Mile Run slag area. He has geological specialties in geochemistry, mineralogy, and field mapping. *572 Briarcliff Rd., Pittsburgh, PA 15221. (412) 624-9320.*

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The **STUDIO for Creative Inquiry** is an interdisciplinary center in the College of Fine Arts at Carnegie Mellon University. Founded in 1989, this center serves as a focus for experimental activities in the arts at Carnegie Mellon. The mission of the STUDIO is to facilitate work in two major areas: artistic creation and the development of educational tools. Within those two categories all work at the STUDIO strives to:

- Bond creative activity with intellectual inquiry,
- Reflect and engage the comprehensive contemporary environment,
- Become manifest through public gestures, and
- Communicate and collaborate with creative inquiry worldwide.

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11 Chester Environmental, "Monongahela River Watershed Act 167 Stormwater Management Plan, Volume 2, Executive Summary," (Pittsburgh: Chester Environmental, prepared for the Allegheny County Department of Planning, adopted by Allegheny County November 1993), page 15.

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Introduction to the Charrette

by Bruce Ferguson

This was the introductory "sendoff" speech for the charrette's participants on the first morning of the event. Bruce Ferguson and Richard Pinkham edited this material to eliminate references to specific visual images (the presentation used many 35mm slides).

Welcome to our people

I can't tell you how happy I am to see these faces here. Some of the people here I know from your work in various parts of the country. Some people are familiar faces from when I used to live and work in the Pittsburgh area some time ago. And for most of the remainder, I am at least familiar with your work from a distance; I have been admiring your work, and that is part of the reason you are here. We have a terrific bunch of people here. I want you to feel very comfortable that you are among a very competent bunch of people.

The nature of the problem of an old urban watershed

We are here to look at an old urban watershed. That is the nature of this problem. It is old, it is urban, and it is a watershed.

The Nine Mile Run watershed is about six and a half square miles in area. In the lower reaches of the watershed, it is occupied almost entirely by Frick Park, so it is an open space. That open space has allowed Nine Mile Run to remain the only free-flowing stream in the City of Pittsburgh. That alone has been enough to allow it to attract a lot of attention, care, and concern for its health.

In its very lowest section, just before it drains into the Monongahela River, the stream drains through an industrial slag dump, very big and visible, right next to an interstate highway. And that has attracted some extraordinary attention. In recent years, that has been the subject of a lot of scientific studies and design proposals. So there is a lot of attention and knowledge about the lower portions of this watershed. There's a lot of thought being given to the possibilities for the future of the lower segment of the watershed.

We are here, in this project, to look at the upper two thirds or three quarters of this watershed. This is the urban portion of it. We are here for the first time to look at the urban portion of this stream, before it discharges and becomes a visible flowing stream on the surface of the soil.

This is a much more complex problem because it is much more than a stream. It is also a system of sewers, and infrastructure, and streets, and houses, and land uses, and economics that are involved with these land uses.

This is an interdisciplinary kind of problem. That explains the different kinds of people who are here. There are physical designer of various kinds — landscape architects, architects, engineers. There are also urban planners; there are also ecological kinds of people; there are also cultural kinds of people here. We are here together to see if we can figure out what are the possibilities for the future for the urban portion of this old urban watershed.

There could not be a more challenging situation. This is a new kind of question. We have spent a lot of time, during my lifetime at least, looking outward to the suburbs, developing new land and extending highways out to it. We've been conscientious at setting standards for what the water quality and so on should be in every new development. But it's so easy to do in new development: it's all on pristine land; all you have to do is set the new standard, and away you go.

Here, we are turning out eyes back to the old urban watershed, where everything is already in place. The systems are already established. This is a technical problem: it involves hydrology and engineering. It is also a problem of human community: the people are already there. When we lay our hands on this watershed, the people are part of the problem, and they have to be part of the solution.

Our objective

Our objective is to illustrate the possibilities for the future, the possibilities for what could be done in this watershed. That is going to take skill and competence and aggressiveness in design, and ingenuity. It is also going to take sympathy and restraint and care in dealing with the human and environmental things that already exist in this watershed.

Our approach to illustrate these possibilities is by designing them. We have people who we know can help us with this kind of problem.

We have four sample sites, which are representative of mixtures of things that happen in this watershed and in old urban watersheds in general. There are different kinds of land uses, but the sites were not selected as samples of exclusive land uses. Every one of them is a mixture of things. It's like turning a kaleidoscope: every time you turn it, you see the same things in a new mosaic.

We have a fifth team, our policy team, which will look at what are the constraints to and opportunities for implementing the kinds of things our designers would like to deal with in old urban watersheds.

We are here to look a hundred years in the future. We are not limited by short-term political feasibility — this is very important. We want to do what is right and what is possible in these places. You competent, ingenious people need to tell us what is right and what is possible, so that policies and laws and initiatives can go in that direction. Laws and policies need to follow what is right and what is possible, not vice-versa. You need to show us what is right and possible so that we know which way to go in the future.

Redevelopment will happen in this watershed. This is one of the few things that we can confidently predict about the future. Times will continue to change, as they have always changed in

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the past. There will be new economic contexts, there will be new populations moving through, there will be new insights that people will have, there will new public initiatives getting old sewers to be dug up and replaced, there will be new kinds of energy sources – long after we are all gone, but it will be happening.

To guide redevelopment, we need to illustrate to future generations what the possibilities are. It will take many human generations to do this. There are thousands of residences in this place; there are thousands of cars moving through it; there are thousands of miles of utilities in here. The process will depend on diverse, incremental, public and private initiatives. You need to show what kinds of possibilities there are for redevelopment for a better old urban watershed in every respect.

Our procedure

We are asking the designers to produce certain kinds of products. The designers are accustomed to what those are. But we have put certain kinds of constraints on them, foreseeing the kind of publication that will be coming out of all this when it is done. These are all listed in the briefing book. Please do refer to the book to remind yourselves as to where this is headed. In the next 24 to 36 hours you're going to be very involved with all the substantive issues on your site: how you are going to solve certain kinds of problems; what are the problems anyway. Just remind yourself of what the implications of this are going to be.

The procedure we're following includes some reporting out to other teams. This whole group will be meeting from time to time so we can inform each other about insights we're having, problems we're having, questions we have, and what some of the possibilities seem to be. We will also be meeting with the public from time to time to inform them about what is going on, and to receive some of their input to what some of their possibilities are, and perhaps what some of their fears are about these things.

We are not left with a lot of design time. The word "charrette" means a short and intense design event. That is certainly what this is. With the amount of time that we have, we will be asking everyone to use the briefing book. A lot of thought was given here as to interpreting the sites: what are the social and watershed issues that exist on your site, what are some of the landscape features that exist there, what are some of the possibilities for design, what are the soil conditions, and so on. These things are all outlined to some degree here.

Use the other kinds of support systems that are around. We have a large library of publications in the other room, which covers a lot of the policy-related things, the ordinances and plumbing codes and so on, that apply to this area, and some of the geology that exists in this area, and the standards and design formulas that apply to stormwater. It's a pretty good library.

One resource that we need to be aware of is the people in this group. Ultimately, every person, on every team, is fair game to be borrowed by another team for an hour or so to answer some technical questions. And we do have, in this room, an expert on local botany and native ecology. We do have a plumbing inspector from one of the local municipalities. We do have an expert on porous pavement. And so on. So when you come up with a specific question, start asking around with me, or Jen, or Tim Collins, or Richard, and we'll try to locate who the people are who might be able to answer your specific questions. They'll be drawn out of the other team for a while, and then they'll have to get back.

Please make decisions fast and aim at the products that we're asking for. We have people we trust to have insight into this and we will certainly be in a very good position for presenting these things on Saturday.

My role is to help make sure people are headed toward the right place, and continue to have the right attitude toward their project. The sort of question like, "is such and such really a part of the problem," or "should we really be considering such and such," or "should we really be headed in such and such a direction." I think the RMI and STUDIO people are relying on me to answer most of that kind of question. Jen will be able to handle all the resource and logistical types of questions.

Each team has a base map which has been drawn. And you have the normal kinds of drafting equipment, for manual drafting. We also have some computers which have GIS and PhotoShop and CAD capability, and a number of computers with word processing capability. We are asking for a narrative summary of what you are producing, in addition to the graphic work.

We have some students, graphic assistants, from some of the universities in the region. Each team will need to assign a notetaker, or perhaps a volunteer can appear. We have not assigned team leaders. There is no designated person at the beginning. Each team might end up with its own dynamic. Perhaps some of them will end up with their leaders; some of them may end up in more of a consensus type of organization. We'll allow that to evolve, each team in its own way.

We're going to take field trips to see your sites rather shortly. Do take with you a copy of the base map. Each team, in going to its site, will have a tour leader. This is not the same thing as a team leader. But each team has a person who is familiar with that specific site, and you should expect that person to introduce you to this place as you are walking around. The policy team will be rotating among all four of the sites.

The nature of the problem of Pittsburgh's Nine Mile Run

Now I want to tell you about Pittsburgh, because people have come from all over the country for this.

The Pittsburgh region

The three rivers meet where the city of Pittsburgh is. Geological studies show where rivers were flowing before the glaciers came along north of here twenty thousand years ago. The Monongahela river was flowing across our watershed, and in those days it was 200 feet higher in elevation than the rivers

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are today. The glaciation knocked the rivers out through the Ohio River, where the rivers go out today, and 200 feet lower. It was a pretty catastrophic event that blasted out the whole watershed, leaving behind the terraces of old rivers which perhaps a third of our watershed is located on. So that is a peculiarity that does not occur elsewhere in the region.

The region is underlain by sedimentary rocks in general. It is full of coal, limestone, sandstone, and shale. Throughout history this has been a center for movement and for the convergence of things. The geography of the rivers and the region they're located in determined that. Most of the people moving west came through this place and got onto their flatboats here to go down the Ohio and up the Missouri. It was very easy to build the railroads along these river valleys, and huge barges were able to move along here. This has been the place where terrific industrial resources, in the rocks and the water and the geography of the place, were converging.

And this is the brilliant aspect of Pittsburgh's history. If you add up all of the industrial mills that are or were in these river valleys, this is one of the greatest industrial complexes in the world. This has been happening during the nineteenth and early twentieth centuries, and is central to what our country has been doing. Pittsburgh is not only a symbol; it is the central physical place where these things have been happening. Many of the mills now are physically gone. The most amazing thing I've ever seen in my life was to see a big Pittsburgh mill torn down, leaving nothing but a flat plain of soil. Pittsburgh needs a new economy now. Some people think that it is going to come from research and development, because the old technical skills that were developed, around the universities and so on that grew up around the industrial mills, are able to keep going in this form. But the times are continuing to change, as they have changed in the past.

The region here has more than a million people. Having lived and grown up in this area I am certain that the people who have lived here and who continue to live here are a generous and hard-working people. They are very proud to be part of this place and of what it is able to accomplish and what it is able to build.

Recent development and its outcomes

The Parkway East going over the Nine Mile Run valley exemplifies the form that we have given development in Pittsburgh and other cities around the United States in recent decades. The interstate highway is taking commuters from downtown and elsewhere, past the old houses and streets, and sending them out to the suburbs. We are turning our backs on the evolution and the care of these old places and getting in our cars and sending people elsewhere.

I know, because I contributed to this in my practice here back in the seventies. People like me have been digging up the old pristine watersheds out in the suburbs and eroding the soil and installing new infrastructure there. And we have been convincing people to move out to these places, in their cars, out to these new, homogeneous widespread places, with the emissions that come out of cars, distant from any kind of resources to support themselves on a daily basis. Pavements are connecting everything together, because we're all automobile-dependent.

And in the downtown areas, our approach has been (at least from time to time; we seem be over most of this stage now) that the old, diverse, human-scale kinds of places needed to be torn down and bulldozed. That was what you should do with old places. And upon their rubble would appear shining new towers, like some of the buildings downtown. It is homogeneous. From all of these buildings people get in their cars and go out to their homes in the suburbs, and the city is no longer functioning as a city.

And Nine Mile Run is left in its culverts and its sewers. At one point, the stream is in a chamber, basically an above-ground culvert. We are driving on top of that thing, we have built right up to it, and it's no longer alive as a stream. The Nine Mile Run watershed has been basically in place, in terms of its urban pattern, for a hundred years and more. I don't think we need to look down on the people who built it the way they did. I think that they solved the problems of their time. They, like we in our daily work, were following the standards of their time. But it is normal for standards to change over time, and for knowledge to increase and for more demands to be placed upon things.

Where Nine Mile Run discharges to an open channel, it suffers from every problem that an older urban watershed could have. When the rain falls, abrupt pulses of floods come through, eroding as they go and bringing sediment. The flood flows get into the combined and sanitary sewers, and they overflow, and raw sewage becomes a large part of the flow. Every rain fall brings oils and so on off of the automobiles, and other kind of pollutants, because this is an old urban watershed — metals and bacteria and so on. Then when the rain is not falling, the base flow is almost nonexistent.

At the bottom of the watershed is the Monongahela River, where Nine Mile Run discharges. The slag pile is nearby. Frick Park is a large open area. The impervious surfaces signify the urban area that we are looking at. What we are looking at is the larger part of the watershed. Nine Mile Run is completely representative of old urban watersheds in Pittsburgh, and in other old cities in the United States. It overlaps four different municipalities. Our sites are in different parts of the watershed. We've got Hunter Park in Wilkinsburg, the Edgewood train station in the center, what we're calling the Gateway site near the edge of Frick Park, and the Sterrett School in the northwest. We are hitting all four municipalities. The watershed area as a whole is over forty percent impervious. There are places in the watershed that are close to a hundred percent impervious.

We know the kind of solution that we have out in the suburbs: large stormwater detention and treatment ponds and wetlands. They can work really well, and look really good, when properly designed and constructed, where there is the space. This is not an option in an old urban watershed, because everything has already been established. Bulldozing and replacing things to install stuff like this is not part of the issue.

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The potential for a redevelopment approach

You can see what they did in the Nine Mile Run watershed, though, a hundred years ago. They took care. They took care of the details where they lived. There are different lanes in the old streets, and different kinds of pavement that occur, with still other pavement surfaces on the sidewalk. The way we take care of an old urban watershed now is going to have to be very careful about details. Everything that is happening in this place is fair game; it is part of the problem. You're going to have to pay great attention to where you are, and what the possibilities are for urban design, and integrating ecology and hydraulics with the way people live in this place, and how to build a city. We need to be addressing all of the folks who live in the houses over all these hillsides. They are part of the issue.

There is a terrific regenerative power in this place. The gray skies that you've been seeing are not just gray skies. They are bringing an awful lot of moisture here, year-round. I'm always struck, whenever I fly in to the Pittsburgh airport, how green this place is. It is trying to regrow; it knows how to make itself healthy. If we would let nature work on our side, and let natural processes of the soil and vegetation and gravity work for us, then we will be able to come up with a healthy city, as healthy perhaps as were the hills before they were developed.

Sterrett School

One of our sites is the Sterrett School, in the City of Pittsburgh. There are impervious roofs and impervious parking lots. One of the strikingly nonimpervious places is the open ground for a small athletic field. There are some streets and houses alongside.

In fact some of the houses are part of our site. The school drains right toward these things, and they all have one culvert in common, where they all discharge. It is one connected place, inside one city block. The school and the houses together are the site. It is a very beautiful neighborhood, very walkable, very safe, with some diverse land uses.

Edgewood Crossroads

At the Edgewood Crossroads site, for those of you who have studied architectural history, the old train station is supposed to be the only station that Furness designed, back in the nineteenth century, this side of the Allegheny Mountains. So it does of course have historic significance. The site has public transportation of sorts going by. There is a lot of traffic on the main road, going a lot of places. A lot of things are converging.

There is a proposal for a new busway along the old tracks, and what they are calling some sort of greenway. I hope that the designers for this site will be inclined to deal with this in a sympathetic way. Residential streets converge from various directions. Farther out there are shopping centers. There is abundant pedestrian traffic. In the minds of the local Edgewood people, there is an extremely unified community, right over top of those old train tracks, and it goes over to the other side. Dealing with the sense of community here, the central intersection in the community of Edgewood, is the urban design problem. And this has impervious surfaces and drainage problems, just like anywhere else. There are all kinds of things that occur here, and every square inch is part of the problem.

Hunter Park

Hunter Park is a local neighborhood park, surrounded by low-income houses. It has not been very well kept. I think that it deserves more public investment. The local politicians are aware that a re-building of the park is probably necessary, and it could very well be done in the foreseeable future. We have some streetscape that can be brought together with the park, so the park and the neighborhood can work together. There is a headwater stream that is passing through. It is in a culvert at the moment. Perhaps there is an opportunity for daylighting.

The local neighborhood park ends up depending on humble kinds of facilities. The little "dolphin" fountain has a great reputation among little kids. When they turn this thing on, the kids are climbing all over it. There is a lot of real community that exists in this place.

Regent Square Gateway

The Regent Square Gateway site is down at the bottom of the urban part of the watershed. The hills are all draining down toward it; Nine Mile Run is down in the valley bottom. The old building on the site, which is now unused, used to be a supermarket. You entered the supermarket from the upper side, or you could get down to the lower part too. This is what we're calling the Foodland site, because Foodland was the name of the old supermarket, but we're also calling it the Gateway site because this old, seemingly abandoned place deserves to have quite a prominent future.

Just on the downhill side of the site is where the discharge from the urban portion of Nine Mile Run happens. All the various culverts are discharging here, including the huge main one. Erosion is occurring. Everything that goes on in this watershed, good and bad, is reflected in this discharge right here.

Down the center of the site is an old city street which was replaced by a new one, when the parkway went through. Tens of thousands of cars are going by here on the new roads every day, in the new system of doing things. The bottom of the old street deserves to be the gateway, because right there starts Frick Park and the open portion of Nine Mile Run and a possible greenway that would connect people not in automobiles but on foot and on bicycles through the park alongside the stream.

Conclusion

Every square inch is a part of the problem and the solution. And everything that happens on each one of these sites is part of the problem and the solution. We want to know: what are the possibilities for the future?

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Hunter Park Team Narrative Summary

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Following is the main written material handed in by the team at the conclusion of the charrette. Rocky Mountain Institute and the STUDIO for Creative Inquiry are grateful to Ken Tamminga for his helpful revisions to portions of this material subsequent to the charrette. The material has been lightly edited and reformatted for consistency with other sections of this appendix.

"What holds people together long enough to discover their power as citizens is their common inhabiting of a single place."
—Gary Snyder, poet

Introduction

As our charrette team broke free from the tight residential matrix, we confronted a surprisingly large and unlikely space. Hunter Park stood mottled in the weak sun of the season, deserted and seemingly unprepared for visitors. Yet it imparted an air of expectancy, as if it were, like a once-gracious tenement, only too ready for re-investment and care.

Hunter Park serves as a major spatial counterpoint to its built context. Hollow and concave in form, it claims the geographical center of its diminutive watershed. Like its surroundings, it is gritty and embattled. On this October day, life is scarce. The scrappy vegetation on its side slopes and its strangely elegant terraces hint at an industrial lineage and resultant "survival ecology" similar to the slag-filled Nine Mile Run far downstream.

From a park designer's perspective Hunter Park is a cache of richly ambiguous potential. Nestled deep in the earthy community of north Wilkinsburg, it tilts deferentially toward the Monongahela River. Despite the impression that this space was formed by the forces of post-glacial hydrology, the presence of water is suppressed, almost questionable. The park's grottoed terraces and tangle of stormwater artifacts at its upper and lower extremities suggest a long history of conflict between indigenous, natural processes and human aspirations.

Clearly, this was once a riparian landscape, bisected by a first order and free-flowing stream. *Hunter Run?* Buried and unmarked conduits, now in disrepair, have long shunted the water away. We trudge past the infamous concrete dolphins, cute repositories of water—controlled, now dry. Toward the park's upper end we rejoice at the discovery of a wetland seep and a few hardy sedges and other wetland obligates.

The *sense* of Hunter Park is that of a place with stories to tell, with anecdotes and allegories that speak of the essence of this patch of Wilkinsburg, and with allusions to distant market, social and political forces. The echoes of those who've used this remarkable landscape resonate back in time—kids' laughter astride the gratuitous squirting dolphins; the crowd cheering a late-inning ball game; the labored breath of miners wrenching fresh coal from the rich seam near the head of Hunter Street. In the more distant past, spring-fed waters trickle clear and light through viburnum and elderberry and sweet birch, bowered above by oaks and the looming, now-extinguished American chestnut.

As much as any locale in the Nine Mile Run watershed, this place has stories to tell.

Historical Overview

Attaining a holistic view of Hunter Park begins by uncovering its chronological layers. In building an understanding of the present makeup of the park—its form and patterns, its uses and problems—the charrette team felt better equipped to create the best possible plan for its future.

Prior to initial settlement, the area was defined by the headwater stream of the Hunter Run sub-watershed situated high in the Nine Nile Run watershed, itself a small part of the vast Monongahela River basin.

As noted in the Chronology below, the lands now known as Hunter Park were transformed over the last 200 years by the clearing of vegetation on the surface and coal mining below grade. A post-industrial culverting of the stream occurred, followed by gradual transformation into a park through the creation of a baseball field and a children's playground. Presently, it serves the neighborhood of north-central Wilkinsburg as a recreational space with still-existing playground, basketball courts, baseball field, and compost piles sitting upon former tennis courts, bracketed by the revegetated side slopes and traditional Pittsburgh-style worker housing beyond.

The pre-industrial landform of the park area was originally a sharply incised headwater stream valley. Like much of the Allegheny physiography, according to Henry Prellwitz, University of Pittsburgh geologist, the landscape was transformed by the industrial development of the Weiman coal mining company. The valley was flattened into several terraces through its use as a dumping ground for tailings, thus diverting the stream around its periphery. What is now Hunter Park was then part of the urban fabric with Coal Street as one of the main access routes to the mining site. The mining industry then brought in a larger working population, evident in observation of maps of this period, showing housing at the base of the site and other surrounding areas. Since this period, the Borough of Wilkinsburg has had a viable Black community and the park area serviced this community in various means such as a playing field for the Negro baseball league in the 1920s and 30s. Plans for future develop-

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ment of the park should relate to the existing Black community and their contemporary relationships to the present site.

A series of post-industrial implementations added recreational facilities and gradually transformed the site into a more developed park facility. This began in the late fifties with the installation of a children's playground and later basketball courts, tennis courts, and a baseball field. It is bordered by a light successional side-slope vegetation fringe, which contains some slope wetland areas toward the upgradient portion of the park. At present, the park is not widely used/visited and is in some disrepair. Stormwater management, ecological and social design issues are numerous.

Chronology

300 million years B.P.–

- Pennsylvanian Period; era of sedimentary bedrock formation, including shale, sandstone and coal
- hot, steamy climate; build-up of swamp material in delta muds forms coal seams over time

100,000 to 12,000 years B.P.–

- lengthy periods of erosion of hardened sediment
- temperate climate vegetation communities form

Post-glacial Era, to late 1700s–

- V-shaped valley of Hunter Park area formed through erosion from melt water runoff from last glacier
- in-migration and inhabitation of native Indian peoples

Pre-Industrial Era, to 1820s–

- larger fauna present: bear, wolves, deer, and beaver
- initial Euro-American settlement: Samuel and Sutia Rippey establish log tavern on nearby Penn Avenue in 1788
- Colonel Dunning McNair lays out first lots, forms McNairsville
- James Kelly becomes major landowner, circa 1800

Agri-Industrial Era, 1825 to ca.1925–

- increasing land clearance for settlers' farmsteads, followed by wholesale deforestation for charcoal production to fuel Pittsburgh's iron and steel industries
- establishment of three main mining companies by the 1860s: Duquesne, Crabtree, and Hampton
- establishment of industrial and land tycoon estates scattered across Wilkinsburg/east Pittsburgh area (Dumpling Hall, Homewood mansion)
- 1887 village incorporated as the Borough of Wilkinsburg
- Coal Street formed initial pathway between coal mines and iron industries lining Monongahela and Allegheny Rivers; site of former homes of miners' families of various ethnicity, becoming predominantly African-American through the early 1900s
- Houston Street serves as original entrance to the mines, skewed from the regular street grid to angle into tailings yard; shanties conform to this peculiar orientation
- "dog mines" developed, so-called because of the dog-and-cart system of transporting loosened coal from the cramped quarters of the Weiman coal seam
- sequence of tailings yards, hillside cartways and related mining works developed in and around site; palimpsest (tracings) still evident on site

Early Recreational Era, mid-1920s to 1940s–

- coal mines largely depleted by the 1920s, with remnant activities extinguished by the Depression
- previously denuded hillsides undergo successional revegetation (benign neglect)
- semi-pro Negro Baseball League–an outgrowth of the Depression and war efforts–adopts the upper terrace for exhibition ball games and practice sessions
- remainder of site relegated to informal open space status

Municipal Park Era, 1950s to early 1990s–

- Wilkinsburg Park System Study (1959) includes Hunter playground and stimulates early park development
- Little League ball field developed in 1970s through sponsorship by the American Legion
- swings installed at mowed area in lower part of park
- Wilkinsburg Recreation Department cited in 1980 for "responsibility for four ball fields" in the area: Whitney, Hunter, upper Hunter, and Green fields
- five-year development plan implemented, 1983-87; generic concrete dolphin fountains installed as part of park renovation
- local but seasonally important activities now include basketball, baseball, and children's play associated with water spouts and play equipment; investment in infrastructure and programming continues to be minimal
- abandoned tennis courts create de facto facility for leaf stockpiling and composting
- informal and illicit activities noted as concerns

Post-Modern Park Era, mid-1990s to present–

- increased interest by surrounding residents in park status (cultural and ecological)
- community leaders, academics and agency representatives begin to see potential for restoration, enhancement, and programming based on principles of identity and legibility, safety, sustainability, ecological integrity and participatory decision-making
- Borough of Wilkinsburg includes Hunter Park in a district redevelopment study
- Nine Mile Run Ecosystem and Infrastructure Charrette represents first major effort to bring together many disciplines and community representatives in seeking common solutions to opportunities and issues of the park and throughout the watershed.

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Ecology and Landscape

Characteristics

Park Terraces. Considered in ecological terms, Hunter Park comprises several distinct ecosystems, all of which can be considered anthropogenic (human disturbed). The largest portion of the park is covered with turfgrass, probably Kentucky bluegrass (Poa pratensis). Soil profiles were not investigated by the charrette, but are likely to be non-original. Almost certainly, much of the leveled area of the park is fill, with sufficient topsoil imported to grow turf.

Original mesic (moist) topsoils in the lower, previously riparian, sector of the site would have to have been stripped during mining operations. Then, to accommodate the tailings yard terraced flats were created, necessitating the burial of Hunter Run. Thus, although spatially reminiscent of a riparian environment, there is no functional riparian ecosystem remaining on site.

Wooded Side Slope Ecosystem. The side slopes, too, have been subject to disturbance since the early 1800s. It is very likely that land clearance, logging, and other activities completely denuded the side slopes and adjacent tablelands. Erosion of topsoil and siltation (and destruction) of aquatic habitat would certainly have followed. Present conditions would seem to support this premise. Species common to stressed and thin-soiled conditions predominate, including black locust (Robinia psuedoacacia), boxelder (Acer negundo) and young Norway maple (Acer platanus). Besides several ornamental trees planted in the playground area, sizable native hardwoods are not present; the larger existing successional trees would appear to be from 40 to 60 years old, coinciding with the closure of the coal mines and the beginning of the current phase of benign neglect. Undergrowth is dominated by non-indigenous, invasive species: privet (Ligustrum spp.), garlic mustard (Alliaria officinalis) and along open edges, multiflora rose (Rosa multiflora) and honeysuckle (Lonicera spp.). Conditions are also appropriate to the aggressive native wild grape vine (Vitis spp.).

Wetland Ecosystem. The other significant ecosystem on site is a small area of wetland just upgradient from the abandoned tennis courts. Using the hydrogeomorphic (HGM) classification system employed by researchers for reference wetlands in the lower Nine Mile Run results in a classification of "slope" wetland, supplied by a mix of ground water seepage and surface water from the valley walls. Soils on this slightly terraced area are continually saturated, and appear to be hydric. Emergent wetland obligate species include sedges (Carex spp.), as well as a healthy stand of introduced teasel (Dipsacus sylvestris). Further investigation is needed to determine if this is a remnant natural wetland, or an unintentional "created" wetland resulting from grading activities associated with mining operations.

The Park as Space. Spatially, the park is quite remarkable. First impressions are that careful consideration must have been given to forming a multi-terraced and generous grotto specifically for park activities. Once one understands the forces of landscape change over the past century or so, it is clear that the park program evolved within the spatial framework carved out through mining operations. The park is, then, a fascinating example of adaptive reuse for recreational purposes, and an early example of the transformation from industrial brownfield to post-industrial civic landscape. Spatial definition and the feeling of being in an immersive, protected landscape is heightened by the openness of the well-connected terraces bracketed by the steeply-wooded side slopes. Visually, the urban residential fabric of the borough is only readily discernible toward the south end. The north, east and west side slopes contain sight lines and effectively block visual access to the adjacent streets and homes.

Issues

Lack of Design. For all of its special qualities and its fascinating heritage, Hunter Park has its share of environmental, programming and design problems, very much a microcosm of its post-industrial urban context. It should be pointed out that the present facilities fairly represent conventional approaches to municipal park installations from the 1940s to the present. Unlike the larger, comparatively well-endowed parks in Pittsburgh such as Frick and Schenley, this park was never designed in a comprehensive manner by a professional landscape architect. It was, rather, fabricated incrementally as demand and recreational fads dictated; the happy marriage of industrial valley form and recreational play fields was not so much preconceived as opportunistic.

Functionality. Functionally, the park has some serious weak points. The edges are ambiguous, lacking clear demarcation and thus sending mixed messages to its various constituencies. There are no formal entry points, although there is some sign of past pedestrian gateways along the east and west brims of the valley. The south end of the park demands the most attention as the socio-cultural focus of the park, but lacks even rudimentary design gestures that might reinforce this as a vital, humane place. Peripheral sidewalks come and go with no apparent logic. Structural maintenance of sidewalks, curbs and gutters is dismal, and weeds and litter line the east and west boundaries. Parking is provided in informal fashion on a graveled lot along the lower west boundary of the park, with ambiguous pedestrian entry in. Internal circulation in the park, again, is informal; pedestrians utilize the asphalt-and-gravel access lane or walk on the turf. Shade tree plantings for human comfort do exist—of particular note are several sycamores—but there is generally very little logic to plantings on site.

Social conditions. Paralleling the physical issues of the park are its social characteristics. Although baseball, basketball and playground activities do take place during the summer season, the park on the whole is underutilized. Past illicit activities have, for the most part, been brought under control, but there is likely some lasting stigma that will take a concerted effort to reverse. Certainly, attention to safety concerns could be heightened as the community begins to re-discover the park. The general level of maintenance and equipment investment is an ongoing concern, indicative of Wilkinsburg's very modest tax base. As such, Hunter Park is not nearly at its full potential in serving

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as the central recreational and civic open space for the residents of Wilkinsburg.

Ecology. As implied above, Hunter Park embraces a fairly typical, and quite dysfunctional, ecology. Invasive species are a major concern, and the overall health of the "forest" is, at best, moderate. From a landscape ecology perspective, a number of park elements—access lanes, grade changes, excess turf, redundant recreational facilities—contribute to landscape fragmentation, resulting in loss of precious interior habitat. Persistent groundwater seeps at mid-terraces points and along the toe of the valley slope have created small pockets of wetlands. These should be protected as locations of high biodiversity, and emulated wherever feasible throughout the site.

Water Resources

Characteristics

Drainage Areas. The Hunter Park Watershed has a drainage area of approximately 60 acres upstream of Maple Street. The watershed (longitudinal) slope is approximately 9% along the main drainage system. Lateral slopes are steep ranging from 20 to 25 percent.

The watershed was subdivided into four (4) sub-watersheds to better define the hydrologic characteristics of the area, as shown on the GIS map in the Hunter Park section of the main report. These sub-watersheds also provided information for locating and sizing stormwater control facilities and features, as described in the Hydrologic Strategies section. Below is an illustration of the impervious areas in the watershed (streets and rooftop areas, as determined by GIS analysis; sidewalks and parking lots not included):

Sub-watershed 1
- Area: 6.0 Ac
- Impervious Area : 1.8 Ac (29.8%)

Sub-watershed 2
- Area: 23.0 Ac
- Impervious Area: 3.4 Ac (14.5%)

Sub-watershed 3
- Area: 6.7 Ac
- Impervious Area: 1.2 Ac (18.6%)

Sub-watershed 4
- Area: 23.5 Ac
- Impervious Area: 2.9 Ac (12.3%)

Hunter Park Watershed (total)
- Area: 59.3 Ac
- Impervious Area: 9.3 Ac (15.6%)

Land Use. The land use in the watershed is entirely residential. The upper portion of the watershed has multifamily dwellings and the rest of the watershed has largely detached single-family dwellings. There are scattered examples of architecturally notable and well-maintained structures near the park, but much of the nearby housing stock is in need of repair. The impervious area (roads and rooftops) cover approximately 9.3 acres or 16% of the watershed.

Issues

Erosion. The impervious areas produce concentrated runoff that has caused some erosion in the upper reaches of the Park and along the steep side slopes.

Drainage. There is no defined channel because most of the area is currently drained by pipes (stormwater and CSO) or by inadequate grass swales that convey the water, but provide no water quality benefit. Most of the drainage system is in need of maintenance. All of the drainage inlets are clogged with sediment and some pipes are broken.

There are inadequate outflow pipes at the base of the watershed. Therefore, there is a need to control the runoff in the park to prevent flooding or erosion downstream. There are cracked concrete sidewalks and curbs at James Street. Several inlets and catch basins in the watershed are full of sediment and debris and the concrete around them is broken. There are several dilapidated housing units built on buried or culverted streams.

Water Quality. There are no water quality BMPs implemented in the watershed. Most of the runoff drains through inadequate sewers or goes into the CSO system, which itself is the subject of agency scrutiny. Even though the imperviousness in the watershed is low, in comparison with other tributaries, the park can provide areas for "treating" stormwater from the residential areas and reduce the amount of runoff. The stream in the lower part of the watershed has been piped, creating flooding problems and destroying the habitat value of this area. Several houses just downstream of the park show signs of water-damaged foundations and related subsidence.

Data Needs. Information is needed on the size and capacity of the combined sewer system. Several assumptions, based on observation, were made regarding this CSO system. In order to finalize the preliminary design illustrated in the hydrological strategies, the impact of an infiltration and detention strategy in the "entire" Nine Mile Run Watershed needs to be evaluated.

There is a need to know the level of pollution that the watershed can "handle" to define the Best Management Practices (BMPs) and policies that are needed. Also, there is a need to understand the hydraulic capacity of the stream (or pipes in the sewered areas) to define the flooding problems and levels-of-service that can be achieved.

Hydrologic, hydraulic, and water quality models of the entire watershed will be useful to understand the contributions of each tributary, as well as the combined impact of the alternative solutions at the sub-watershed scale. These models can also be used to identified areas with significant erosion and flooding problems and to evaluate alternative solutions to these problems. In addition, models can be used to understand the relative pollutant load contributions and target BMPs in specific sub-watersheds. These models can also be used to design BMPs and

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make sure that their interactions are beneficial for the watershed.

In order to prioritize problem identification and evaluation of solutions, watershed goals need to be developed. These goals should reflect public values and should be based on sound science. Implementation will be successful if the relationship between public values, science, and policy is understood and integrated under a watershed management framework.

Strategies

The overall strategy for the Hunter Park watershed considers the area's natural and cultural history as a framework for identifying opportunities to resolve drainage and erosion problems, improve water quality in the watershed and, as importantly, provide new life for Hunter Park as a civic open space. The following strategies are used to illuminate these environmental and historical features and to improve the health and social vitality of both the park and the watershed as a whole.

Improve Runoff Quality

Filter, detain, and infiltrate runoff to remove pollutants, recharge the aquifer, and reduce combined sewer contributions. The components of this strategy include:

- **Created Wetlands.** Create wetlands that collect water at the toe of the slope in the upper terraces of the site. Plant with native emergent and scrub-shrub plants, varying microtopography using 6-8" undulations to enable an intermixing of obligate and facultative species. The wetland is used to filter pollutants (i.e. suspended solids, nutrients, metals), reduce peak flow rates, stabilize the flow of water into the grass swales described below, and provide a modicum of habitat. Created wetlands are located at the bottom of sub-watershed 4, treat runoff from approximately 30.2 acres, and have a hydraulic treatment capacity of 25,000 cubic feet.
- **Woodland Bioretention.** Create a woodland bioretention area consisting of sand and soil mixtures planted with native plants. A pretreatment / sediment decanting area is provided in the upper part of the bioretention area to dissipate energy and collect the coarser sediments, thus enhancing water quality through to Nine Mile Run. The bioretention area is located in the outlet of subwatershed 3 and treats approximately 6.7 acres. It has a hydraulic treatment capacity of 10,000 cubic feet.
- **Enhanced Grassed Swales.** Form vegetated swales with infiltration and filter zones (sand and topsoil areas 1 - 2 feet deep and 10 - 15 feet wide) that filter pollutants as stormwater moves through the site. These swales carry the water around the ball field and through the lower part of the park. They are the main treatment method for water from sub-watershed 2 (an area of 23 acres), and have a hydraulic treatment capacity of 20,000 to 30,000 cubic feet.
- **Disconnected Roof Rainwater Leaders.** The residential areas in the upper portion of the watershed and along Coal and Swissvale Streets will have a program to disconnect the roof leaders from the CSO system.
- **Intercept Street Runoff.** Some of the roof runoff and most of the street runoff is conveyed through curb and gutter and inlets to the drainage/CSO system. This component will include disconnecting the streets from the drainage/CSO system and conveying them to the park BMPs for treatment. Natural stone energy dissipaters will be used to prevent erosion problems at the outlet of the conveyance systems, or natural swales will be used with small check weirs.

Methodology. The water quality volumes were calculated using EPA's simple method and were based on a 2-year, 24-hour storm. A groundwater recharge volume was calculated using the methodology suggested in the Pennsylvania Handbook of Best Management Practices for Developing Areas.

Stream Restoration

- **Stream daylighting.** Daylight the stream at Hunter Square and repair the inlet and pipe at James Street. The restored stream will convey stormwater and will provide an opportunity to recreate the native stream habitat. The stream also serves as an amenity and focal point for the park, revealing long-buried hydrological cycles.
- **Investigate and rectify sanitary sewage problems** in the area of Hunter Park. Substantiate location of, and resolve, any existing illegal sanitary line connections into the main storm culvert. Assess the condition of sanitary and combined sewers, and repair as necessary.

Methodology. A bioengineering approach was used to size the stream meanders and to protect the banks. The peak discharges were computed using a Corps of Engineers hydrologic model and verified by using the rational method. Manning's equation was also used to verify that the channel will convey the flows from a 2-year and 10-year storm. These calculations are preliminary in nature and should be verified as the projects moves to implementation. In addition, the capacity of the downstream drainage system needs to be determined, and the interaction of the sanitary sewer systems with the drainage system needs to be defined.

Urban Landscape Improvements

- **Work with culturally-related patterns of the site.** Embrace and strengthen the wonderful spatial qualities of the park. Enshrine the sequence of sub-spaces that give the park its inherent logic: intensely social landscape at the south end, recreationally-expansive mid-terrace area, and naturalistic zones of the north, west and east valley walls. Re-cast old pathways (such as the cart path traversing the north valley slope), and tell the stories that go with these special features.
- **Incorporate local materials** that elicit linkages to the site's cultural and natural history. For example, use red

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sandstone as building material for gateways and path edging. Sturdy and simple iron hardware would fit well into the park's architectural palette.

- **Reduce road widths and detain/infiltrate runoff.** The road width on Coal Street will be reduced along the park and parallel parking will be incorporated. The road width reduction will reduce the overall impervious area. The parking stalls will have a permeable paving system that will collect and treat/infiltrate the runoff from the streets and upstream residential areas. Ground water recharge and pollutant removal will be enhanced with the use of this component.
- **Plant street trees and add a new public square to the park.** In addition to the air/water quality benefits of trees, the square will provide opportunities for enhanced cultural interactions.
- **Reduce paved areas.** Reduction of impervious areas will increase groundwater recharge, slowing storm surges and enhancing water quality down-gradient.

Habitat Enhancements

Aquatic and terrestrial habitat will be greatly enhanced by the implementation of hydrologic strategies. However, the park's ecology should not be considered just a hydrological spin-off. As the preeminent open space in Wilkinsburg, situated at the height of land between the Monongahela and Allegheny Rivers, the park has great ecological value, serving as a sink for biological diversity to the surrounding area. Following are some strategies to enhance the ecological integrity of the park.

- **Ecological Inventory.** A detailed and long term study of the wooded portion of Hunter Park will more clearly assess levels of ecological integrity and will provide a basis for ecosystem management approaches. Consider the approaches and resources that may be available from the Carnegie Museum of Natural History.
- **Woodland Management.** A strategy to control invasive species, build soil resources, and restore indigenous vegetation communities is vital to the long term ecological health of the park. A multi-disciplinary approach should be instigated, drawing from both science (forestry, ecology) and design disciplines.
- **Maximize habitat contributions of all installations.** Hydrologic and infrastructural improvements should be multi-faceted in terms of cost-benefit, and should seek to create ecologically-functional habitat wherever possible.
- **Minimize energy and chemical inputs.** Adopt BMPs relating to sustainable park management practices. Keep manicured turf to a minimum, and select organic maintenance methods. Restrict the use of fertilizers and pesticides. Plant native species that suit the conditions on site and require minimal watering and tending.

Cultural History and Public Involvement/Education

The above strategies were developed using natural and cultural history as a framework to identify structural and biophysical opportunities for park and watershed enhancement. Equally as important to the residents of Wilkinsburg, the following strategies help preserve and celebrate the rich history of the area, and provide opportunities for public involvement and education.

- **Enhance the safety of users.** Incorporate lighting in high-use areas, and install other safety measures. Monitoring and security of the park is essential in ensuring that it become a cherished, well-used facility serving a diversity of residents.
- **Locate existing cultural events and watershed festivals in Hunter Park.** Besides the obvious community benefit of coming together in a central green space, special events and programs can be used to promote ecosystem stewardship and help citizens to identify with the watershed. The improved access to the park will promote its use and further enhance opportunities to learn about the hydrologic strategies through interpretative signs and guided tours.
- **Engage local skilled and unskilled labor,** *and use volunteers* in regenerating and managing the park. Nearby residents who not only gain some income from park projects and program, but also invest time and sweat, will undoubtedly be concerned about its future well-being. The Citizens' Advisory Committee for Shade Trees may wish to take on some role in revegetating the park, under professional guidance.
- **Take advantage of existing agency and non-profit group programs.** Inquire into the ability of Economic Development Group East (EDGE) and similar organizations to assist in the park's improvement. Consider win-win propositions such as the Youth Build program, part of the AmeriCorp initiative. Youth involved in the park's built projects receive not only on-the-job construction training, but in turn receive scholarships to attend college.

"The Hunter Park Story"

There once was a stream that flowed quick and clear. It linked the hills of its place with the mighty river meandering far below. Through time the stream bit deeper into its native earth, forming a sharp incision which invited the trees to bend inward. Peaceful people came to live and pass through the valley floor.

The Weinman brothers saw the black slice along its banks and lived for many years off the resources that the land offered. Coal became king, and the stream disappeared. Workers swung picks and dogs pulled coal. The land was stripped of its cloak.

With the loss of form and mineral substance, the land lay fallow. The detritus of profit and greed would forever alter the stream's place. Economic decline and mobility brought both despair and opportunity to this place. Who knew that it would be a baseball and bat in the hands of Black people that would make this place resound with life?

The Negro League thrived for years, and reclaimed the altered

landscape, proving its worth for the people. With the vanishing of the league the place was remade; kids haunted its young woods above a stream long buried. Authorities answered the landscape's unmet potential with redevelopment.

A team of specialists came to town and saw a stream as local leaders promoted new life for the park in the guise of a new baseball park. In partnership with the pro league, they envisioned a place where ball, stream and wood merged.

Hunter Park, Hunter Wood and Hunter Square together formed a place of past, present and future.

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Edgewood Crossroads Team Narrative Summary

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Following is the main written material handed in by the team at the conclusion of the charrette, lightly edited and reformatted for consistency with other sections of this appendix.

Introduction

The land planning community across the U. S. is quite taken with the concept of the "village"—creating new mixed use communities at every roadway intersection, while preserving the surrounding land. Edgewood is just such an "urban village." Formed in the later part of the 19th century and centered not on a roadway intersection but a rail line right of way, this village evolved as a reversal of the current planning model. The village, complete with community features such as schools, government centers, markets and transportation, was well separated from the then-distant urban center.

The train station is a focal point of the village, and while the relationship to current stormwater management is somewhat unclear, the structure and surrounding space is clearly the heart of Edgewood, and therefore the symbolic heart of the study community.

The study area known as the "Edgewood Crossroads" is in fact much greater (and more complex) than the immediate vicinity of the referenced building. The specific structure, the station, is of both historic and cultural interest, and is located at a low point in a small catchment where stormwater has impounded during recent storm events. Local people identify this flooding as impacting the potential restoration of the train station. Many are also deeply troubled by planned improvements to the railway corridor which the building originally served. That set of issues, while not directly related to stormwater management or the associated water quality issues of stormwater discharges to the municipal sewer system, is nevertheless perceived as a part of the problem and must be addressed in this site analysis.

This analysis deals with stormwater in the larger contexts of Pittsburgh and the complex social, environmental, economic and historic determinants that have shaped the city. Our approach is accordingly based on the following principles that provide an understanding of the interrelationships between water systems, urban design, and social values and how these can provide Edgewood with appropriate and integrated solutions.

Principles of Stormwater Design

- Water systems are an integral part of community affairs and should be a part of the life and built environment of Edgewood.
- Alternatives to stormwater systems should be a visible and tangible part of the urban design framework of the town.
- Every solution to a stormwater problem should have three or four potential benefits.

Town of Edgewood

The Borough of Edgewood has a rich history and strong sense of community pride, and is comprised of some 600 families who occupy a portion of the Nine Mile Run watershed. The original settlement was separated from the city of Pittsburgh, and connected by the rail system which still bisects the community. In a regional context, the Borough is located in the east-central portion of Nine Mile Run, and includes several sub-watersheds which were originally formed as perennial open channels, draining west to the stream. Major sections of these small streams have long since been placed in pipes and culverts, with the low areas filled and graded. In virtually all of Edgewood, little remains of original stream channels or riparian areas, with the exception of a few pockets of wetlands and remnant streams.

For the location of concern, the area adjacent to the train station receives stormwater runoff from an upland catchment of some 73 acres. All of this drainage area is within the northeastern portion of Edgewood, and is almost totally built, with the higher grounds comprising a community of relatively large, single-family Victorian-era homes. The lower portion of the small drainage area is occupied primarily by institutional and a limited group of commercial buildings, centering on the train station. In fact, the station provides a real center to the community, and links all quadrants of the community in a political sense. The topography is steeply sloping, and virtually all roadways are sloped greater than 8%. In fact, the residences, like most of the Nine Mile Run watershed and the entire Pittsburgh region, are situated on steeply sloping parcels. These specific homes are of much larger size, both in terms of structure and lot size, than most of the balance of Nine Mile Run. They comprise the more upscale portion of the area, both economically and demographically. The following reflects those community values that are relevant to our study.

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Community Values

- Maintain social and physical sense of community;
- Preserve, maintain and expand green spaces;
- Reinforce pedestrian connections; and
- Utilize rail station as a point of community focus and pride.

Wastewater Conveyance System (Sewers)

All of the 73 acres which drain to the train station location are served by a sanitary sewer system, which conveys wastewater down the valley to the regional collection system. In fact, some residences situated outside the topologic boundaries are included in this service area. While stormwater from road inlets are not added to this separate sanitary system, a number of roof drains from individual residences have been connected to the sanitary sewer lines, and produce what is known as illegal inflows to the wastewater system. This problem is well documented throughout the watershed, and occurs on many homes, both large and small. Given the age of these residences, the practice of connecting roof drainage to the sewer line in the street was quite common, long before the current wastewater treatment system was constructed at the downstream end of the pipes. Early development practices thought nothing of discharging both household sewage and unwanted runoff down the slope to the nearest stream, usually in the same pipe. For the purposes of this analysis, it is assumed that approximately 50% of the residences include at least partial roof runoff discharge to the sanitary sewers.

Stormwater System

Storm sewers serve some portions of the catchment, with inlets located along much of the roadway network. Most of the current roadway runoff includes lot and rooftop runoff discharges, which are conveyed to the curb line or run overland to the street. The actual surface area which drains to the train station location is reduced by the diversion of some storm drains to the adjacent watershed, and it is unclear exactly how well the surface waters follow this pattern or bypass surface inlets to run directly downhill, especially along the curbs of Maple Avenue. For the purpose of this analysis, the full 73-acre area is analyzed as a single problem, even though the boundaries vary with both sewage and stormwater drainage areas.

The catchment is comprised of about 10.1 acres of rooftop impervious surface (about evenly divided between residential and institutional rooftops), with an additional 6.4 acres of roadway, for a total GIS-estimated impervious surface of 16.5 acres, or 22.5% of the catchment. Adding a roughly and conservatively estimated 2 acres of private drives, parking lots, and sidewalks brings the total impervious surface to 18.5 acres, or 25% of the drainage. The remaining pervious area of 54.9 acres is largely in trees, lawn and assorted ground covers within the residential portion, and in maintained lawns or recreational fields in the institutional lands. In both cases, the landscape reflects a significant amount of land alteration over the period of occupation, especially grading of flatter areas for public spaces or private patio or lawn areas.

Design Goals and Investigation

The primary design goal is to provide stormwater recharge for the 2-year frequency storm event, which for the Pittsburgh region is 2.5 inches of rainfall in 24 hours. This is a fairly straightforward criteria with new development on soils which have reasonable drainage properties, and sufficient site space is available to integrate such a stormwater management system with site demands and constraints. In a catchment which is totally built and any solution must be retrofit within the existing framework of buildings and surface features, it is no simple task. For this specific area, the relatively steep slopes, and the positioning of structures on the landscape further complicate the criteria. In fact, it is apparent that any stormwater recharge/infiltration system proposed would be quite different on the residential parcels than what might be feasible on the institutional parcels. In addition, the physical properties of the soil mantle are far more favorable at the lower end of the catchment, on the ancestral Monongahela River terrace, with thinner soils on the upland residential area.

Following field investigation and examination of topographic conditions, it was concluded that the open spaces in the lower portion of the area, surrounding the institutional buildings, offered significant potential for groundwater recharge beds. These potential beds cover some 6.2 acres, not including any grassed or open areas directly surrounding the buildings themselves. With the installation of sub-surface storage and groundwater recharge beds, these beds could provide a significant reduction in stormwater runoff from the catchment. If a bed depth of 18 inches were provided in a uniformly-graded aggregate bed (40% storage volume)—or an equivalent storage volume provided by pre-formed chambers—the entire rainfall runoff of a 2-year frequency storm collected from an area of 18 acres could be infiltrated. (This assumes that 0.5 inches of precipitation are infiltrated by these beds during the course of the 2-year, 24-hour storm.) Thus the potentially available lands could provide for infiltration of much of the runoff from impervious surfaces uphill. As this stored runoff slowly percolated into the soil mantle beneath the play filed or lawn, it would recharge the groundwater and slowly drain down-gradient beneath the surface toward the valley.

It is unlikely that all of the potential beds could be made available, but the areas surrounding the structures could also serve this infiltration purpose, if trenches and beds were carefully configured. The dimensional area of this land is not developed here, but clearly would be tied to specific design solutions for individual buildings.

Within the residential area, the proposed recharge solutions take the form of specific designs for each lot or residence. It is possible to develop several generic solutions, based on typical configurations identified in the area. For the larger residences, fairly large front lawns and rear yards are common, although slope constraints may limit recharge opportunities, especially in

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rear yards. The design criteria applied in all cases is to store and infiltrate the total rooftop runoff for the 2-year rainfall (2.5 inches), and without estimating the specific infiltration capacity of the soil, a required bed storage of 2 inches per roof area is used. It is also assumed that the typical large residence has a footprint (and corresponding roof area) of 2,500 square feet, and that half of this area drains to the front and rear yards, respectively. Thus a volume of 208 cubic feet of runoff would need to be stored in a bed or trench, or in a surface depression if necessary, for each side of the house. The actual sub-grade volume would depend on the use of a stone bed or a manufactured void chamber (Infiltrator or equivalent) installed in the bed area.

Drawings from the charrette team illustrate examples of these designs. If we are successful in infiltrating the rooftop runoff from all residences, we will not only create a significant reduction in runoff, but will also reduce the inflow of this stormwater to the regional sewer system. If the inflow is estimated to be from 50% of the residences, or 2.5 acres of rooftop impermeable surface, the annual precipitation of 41 inches on these surfaces will be removed, or a total of about 372,000 cubic feet per year (2.78 MGY). The net economic benefit of this removal of inflow translates into a potential annual cost savings of some $5,500—based on unit treatment cost of $2 per 1,000 gallons.

In terms of the total wastewater flow from this service area, the estimated existing sewage flow is about 54,000 GPD, or 19.7 MGY. Thus our removal of inflow represents about a 14% reduction in flow.

Concept Plan

The Concept Plan provides a design solution to the impact of stormwater on the region, community, and individual by responding to the following community issues while respecting the design principles and upholding community values defined earlier.

Community Issues

- Storm flooding of the train station intersection.
- Impact of runoff on residential sandstone basements.
- Town non-compliance with Federal Water Quality Standards.
- Lack of public consciousness regarding the value of water.
- Increase of non-pervious surfaces is occurring without impact analysis and public input.
- Public inspection and disconnection of downpipes from sewer system is imminent and without a plan for resident remediation.
- Street tree removal for overhead utilities is occurring without regard to multiple impacts on community.
- Threat of losing train station as a community symbol of pride and focal point.

Stormwater issues and potential solutions can be expressed with urban design, integrated within the social focus of the community. The station area is a natural gathering place and park, and the proposed site design integrates these functions with a form that offers a stormwater management demonstration facility as an inconspicuous part of the design. The central space will be a depressed bowl, with a porous bottom which will both retain and infiltrate stormwater. The surrounding wall will be both sitting space and berm, serving as a gathering place during dry weather.

During rainfall, the depression (more a forum with a tiled geometric or paver block bottom) would collect surface runoff from the surrounding plaza, filling and then slowly draining over a one or two day period into the sub-surface by an infiltration bed beneath the site. This would only take place some 30 days a year, and for the balance the space would provide communal gathering and play, as a celebration of both the village and the transportation connection, as well as a working example of stormwater management. In this context, the site would serve as the gateway to the Transit Greenway and Trail, to be developed within and as part of the new transit corridor. The recharge system would continue the educational role of the village center, almost as a celebration of stormwater, rather than just a problem solution. It would become a part of the urban fabric, illustrating how to manage runoff while preserving the form and function of the village.

The concept of the urban village should also be reinforced by re-introducing the urban forest, which provides a number of related environmental and aesthetic benefits, as a modifier of urban climate, improving air quality and reducing noise (especially train and bus background noise). In a very practical sense, this new woodland will also reduce the amount of rainfall runoff generated from the landscape, and is totally consistent with the original mission statement for the Nine Mile Run stormwater management design charrette.

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Sterrett School Team Narrative Summary

Charrette Team:

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Following is the main written material handed in by the team at the conclusion of the charrette, lightly edited and reformatted for consistency with other sections of this appendix.

Sterrett School Site

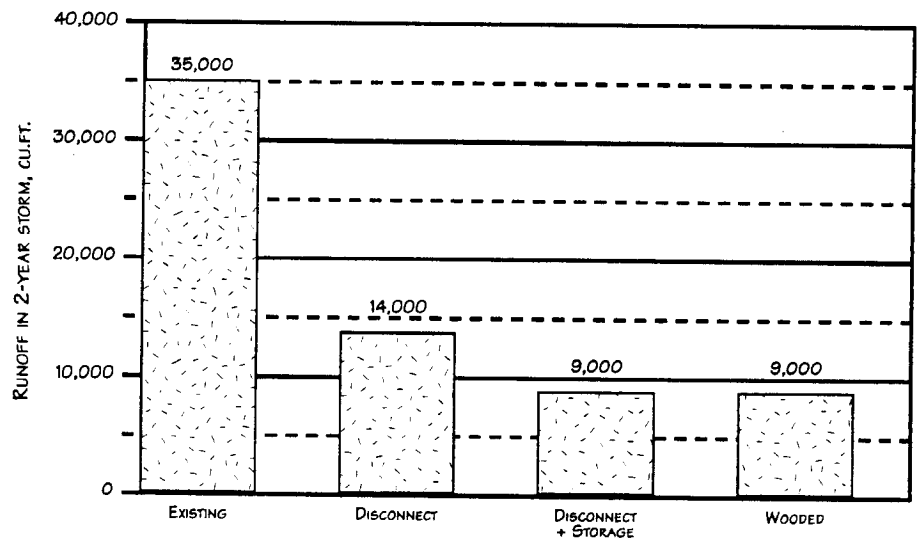
The study site of Sterrett School and eight residences is 5.1 acres, and sits at the headwaters of Fern Hollow. This headwater sits upon ancient, elevated sand and gravel deposits. A geologic finger of sandstone juts into the study site. It is in this area that the original headwater stream, formerly known as Salamander Creek, fed into Fern Hollow. This channel was subsequently filled and houses constructed. The lowest grade on the study site is an alley servicing 3 homes and 3 garages, with adjacent stormwater inlets that occasionally back up and flood. A 36" combined sewer traverses underneath the alley, following the course of the original streambed. The main drainage issue is that some of the homes experience problems with water in their basements.

The school and the surrounding residential area consist of impervious surfaces such as roofs, sidewalks, and roads, together with more pervious areas such as lawns, bare ground, gardens, etc. The majority of our time and effort was directed to the school property. The project goal is to retain or eliminate the volume generated by a 2-year, 24-hour storm event (2.5 inches) from entering the combined sewer system.

Summary of Hydrologic Results

Using the rainfall/runoff relationships from TR-55, 1.8 inches of runoff are now generated from the grass and residential areas, and 2.5 inches are now generated from the impervious areas. The resultant volume that now enters the combined sewer system is approximately 35,000 cubic feet. The estimated runoff from the site prior to development was 9,000 c.f. The major strategy proposed to reduce the 35,000 c.f. volume to 9,000 c.f.s. is disconnectivity from the sewer system. Using this principle, the volume of water entering the sewer system would be 14,000 c.f. This would be primarily accomplished by restoring the natural drainage, then disconnecting the roof leaders, and encouraging infiltration through sheet flow. In order to reduce this volume of water closer to the pre-development level, an additional 5,000 c.f. of water need to be retained or infiltrated (14,000 c.f. - 9,000 c.f. = 5,000 c.f.). BMP practices are proposed to approach this reduction. Figure 1.0, Summary of Runoff Volumes, is a summary of the amount of runoff for each site condition.

Figure 1.0 - Summary of Runoff Volumes



The hydrologic result of "disconnects" and stormwater storage at the Sterrett School during a 2-year, 24-hour storm.

Recommendations

A summary of our ideas and recommendations is presented below.

Retention/Reuse/Infiltration on Site.

A. The roof of the school represents a major impervious area (16,000 square feet). Our plan is to collect this water in a ganged series of tanks or cisterns and then develop a cascading system of priority usage such as:
   1. Irrigation of the gardens and ball field;
   2. Indoor use to flush toilets, for cleaning, etc., and
   3. Retention in bladders in the attic of the school to help moderate temperatures in the building.

   We estimate that approximately 3,000 c.f. of water from the roof would be retained using these techniques. Overflow from the cistern system would flow along an "art

creek," into a infiltration/bioretention swale, and eventually into Fern Hollow.

B. Replace current impervious paved surfaces with more pervious/porous pavement surfaces. Possible areas include the parking lot, the asphalt playground area and the sidewalks.

C. Improve the permeability/retention of the lawn and ball field area. Ideas include bioretention areas under the field, wetland or swale areas around the field, and terracing all or part of the area where the cobblestone wall currently exists between the ball field and the lawn/asphalt playground.

D. A tree planting program would reduce runoff by encouraging infiltration, capturing rainfall, and modifying area air quality and temperatures.

Re-Introduction of Water to Frick Park.

A. Remnants of the original stream system running into Fern Hollow cross a portion of the study area east of the ball field, where private homes are located near the intersection of South Homewood Avenue and Edgerton Avenue. Long-term residents were interviewed who remembered a creek, called Salamander Creek, with a bridge. At the present time the road acts as a barrier to the movement of water across the road and into the ravine of Fern Hollow. Historic maps show the pre-existing water flows and geology. Serious consideration should be given to enhancing the movement of water into Fern Hollow to restore some of the water which now passes through the park in combined sewer lines and is not available to the park ecosystem. This water causes flooding problems in the basements of the homes in the low areas. The origin of this water is not clear and there seems to be some confusion among the neighbors and a tendency to blame each other or the school for the water problems.

A range of solutions was considered. Our preferred solution would be to close off South Homewood Avenue and re-grade the existing raised area to reestablish the natural drainage at the site. A pedestrian bridge would be built to allow access between Frick Park and Homewood Cemetery. The water would enter the ravine via a cascade outfall with a terraced slope, geotextile and boulders. This outfall is fed by a infiltration/bioretention swale which collects drainage and cistern overflow from the school site. The length of the infiltration/bioretention swale is approximately 500 feet. Using a width of 4 feet, a depth of 4 feet, and a storage ratio of 0.35, the resultant gravel storage is 2,800cf. An additional channel space created by gently sloping banks above the gravel surface would fill up to 6 inches deep and 12 feet across during large storms, providing an additional 2,000 c.f. of storage during flow. Ideally, we could combine this concept with the "art creek" on the Sterrett School property. The stream could flow above ground on the school property and then connect to above ground or underground flow across the residential area. A short term solution would be to develop a system which would allow water to drain under the existing road and into the ravine.

Residential Areas

A. The surrounding residential/commercial area consists largely of homes and small businesses (estimated 20% impervious surfaces). Less discussion was given to these areas; however, it is suggested that the school could serve as a model or demonstration area for techniques to retain/infiltrate water that could be applied or modified for residences. Suggestions include the following:
   1. Disconnect the roof leaders so that water is retained in cisterns or water barrels, used to water lawns and gardens or create water features on individual properties.
   2. Increased planting of trees in this area.
   3. Replace sidewalks, driveways, etc. with pervious surfaces.

Education

We believe that education is a key feature of any plan for this area. Our broad goals are to educate (provide environmental education to students and neighbors) and emulate (set an example/model for actions to address the problems).

A. Sterrett School
   1. Connect educational objectives/curriculum of Sterrett School to the Nine Mile Run watershed. According to the "Proposed Academic Standards for Environment and Ecology," developed by the Pennsylvania Department of Education (9/1/98), applicable standards for 7th grade (Sterrett is a middle school) include "acquire the knowledge and skills needed to: ... explain the role of the water cycle and understand the role of the watershed."
   2. The "art creek" is envisioned as a mosaic, student-designed project which would provide artistic expression, and information on the watershed and native flora and fauna. The flow-way would carry water during rainfall events and remain dry at other times. This feature could be used to teach about watersheds, impacts of urbanization, etc.
   3. The home economics teacher who oversees the herb garden and the school's student "green team" expressed great interest in having a greenhouse on the school property as an important teaching tool. The greenhouse could utilize water collected from the impervious surfaces. In addition, the greenhouse could be shared by the Studio for Creative Inquiry at Carnegie Mellon University which is also interested in greenhouse space.

B. Community
   1. It is critically important to educate and inform the general public, using some education goals set forth by the EPA. We support the following:
      a. Environmental, economic and social/cultural benefits and burdens are distributed fairly among all members of the community;
      b. Community members have equal access to an opportunity to participate in community decision-making processes;

c. Community activities do not unfairly impact people or the environment in other communities near or far, nor are these impacts passed on to future generations;
d. Terrestrial landscapes range from the smallest mapable landscape or ecotope (biotope) to the ecosphere, the largest tangible global landscape system of our total human ecosystem, which can be viewed as the highest organizational level of the ecological hierarchy, integrating human society and its total natural and physical environment.

2. We also believe that people living in the upper portions of the watershed should understand the concept of a watershed and the impact of their activities on the lower portions of the watershed. We hope that these people will take ownership of the watershed.

Hydrology and Hydraulics

The following section details the procedures and results of the hydrologic and hydraulic analysis. The project goal is to reduce the loads on the combined sewer by retaining or eliminating the volume generated by the 2-year 24-hour storm event (2.5 inches) from entering the combined sewer system. The hydrologic analysis includes the site before development as a school and residential area, the existing conditions, and the proposed improvements. The site area is defined as the Sterrett School, the residential area to the east, and one-half section of the surrounding streets. The site area is approximately 220,000 sq. ft. Due to the time constraints, nomographs from Technical Release 55, Urban Hydrology for Small Watersheds (TR-55) were used to analyze the runoff depth (volume) for each condition.

Pre-Development Hydrologic and Hydraulic Conditions

The pre-development land cover condition was assumed to be woods in fair condition. Due to the lack of soils information, "C" soils were used in the analysis. This is justified due to the stony conditions, clay content, and shallow soil depth described in the project conditions. This represents a Runoff Curve Number (RCN) of 70. The resultant runoff depth, based on a 2.5 inch storm event, would have been approximately 0.5 inches. The total runoff volume for the site would have been approximately 9,000 c.f. (0.5 inches x 1/12 feet per inch x 220,000 sq. ft.). The geologic maps of the area indicate that a stream channel traversed the site. This channel was located in sandstone, with surrounding alluvial soils.

Existing Hydrologic Conditions

The site is divided between a school and single family residential land use. To be conservative the RCN values were modified to account for wet (ARC 3), or saturated conditions. The composite RCN values for the school site were based on the impervious area and grass in fair condition. The RCN used for the residential area was 90. Using the rainfall/runoff relationships from TR-55, 1.8 inches of runoff are generated from the grass and residential areas, and 2.5 inches are generated from the impervious areas. The resultant volume is approximately 35,000 cubic feet.

Existing Hydraulic Conditions

The channel which traversed the site has been filled in and houses have been built on top of the fill. A 36" combined sewer now runs along the channel. The site is surrounded by a series of inlets, which drain to the combined sewer system. This system becomes very deep as it outfalls into a collector sewer in Fern Hollow. A small trench drain is found in the alley that connects to South Homewood Avenue. This inlet and the inlets along the low point in South Homewood Avenue frequently clog, as they are undersized for the existing flows from the site and from contributing areas uphill. The roof drains from the school are connected to an underground system that feeds into the storm drains. The front of the school sheet flows to South Lang Avenue. The parking lot drains to an inlet that outfalls into the drainage system in Edgerton Avenue. One-half of the rear of the school drains to Edgerton Avenue by a concrete channel. The other half drains to a concrete channel and outfalls to the drainage system in Reynolds Avenue. The northern half of the ball field drains to a channel that runs along the east school property and then down the alley to the trench drain. The rear of the residential area drains to the trench drain. The front of the residential areas drains to the streets.

Retrofit Program Hydraulic Elements.

The key element of the retrofit is the disconnectivity of impervious areas to the combined sewer system. The two major mechanisms for accomplishing this are the termination of South Homewood into two cul-de-sacs, and the incorporation of cisterns around the school building. The termination of the street allows for sheet flow and channel flow from the school and residential areas to flow into Fern Hollow Creek. The swales that collect water from the school and residential areas would be modified as dry swales, with infiltration trenches. This will allow for additional recharge into the groundwater. The cisterns will be used for greywater in the building and irrigation of the ball fields. Overflow from the cisterns during events greater than the 2-year event will drain into the infiltration/bio-swale. Additional bioretention cells will be used on-site at the terminus of the parking areas and to capture sheet flow from the sidewalks and outdoor asphalt play areas. The runoff potential will also be reduced by a tree planting program.

Retrofit Program Hydrologic Analysis

By restoring the natural drainage pattern, only about 10% of the site will drain into the current storm drainage system (the combined sewer). This includes one-half of the remaining street section, the parking area, and the northwest portion of the school site. The resultant runoff, using ARC 3 is approximately 14,000 cubic feet. The estimated difference in runoff volume that will enter the combined sewer from the wooded condition

to the retrofit condition is approximately 5,000 cubic feet (14,000 c.f. - 9,000 c.f.).

Structural BMPs Control Analysis

The length of the infiltration/bioretention swale is approximately 500 feet. Using a depth of 4 feet with a width of 4 feet and a storage ratio in the gravel of 0.35 the resultant storage is 2,800 cubic feet for the infiltration area. Above the gravel, an additional 6 inches of "ponding" during flow in a vegetated, gently sloped trapezoidal channel averaging 12 feet in width provides 2,000 cubic feet of additional storage. Because of the sandstone substrate, infiltration is assumed. An underdrain system, which releases the discharge past the peak of the storm, could be utilized if infiltration is infeasible. The infiltration/bioretention swale decreases the volume another 14%, to meet the 9,000 c.f. requirement for pre-development runoff.

The above calculations do not account for additional reductions due to reforestation, mulching, impervious pavements, and bioretention cells.

Summary of Hydrologic and Hydraulic Analysis

By incorporating features into the retrofit that utilize disconnectivity, the load to the combined sewer has been significantly reduced. Additional techniques to encourage infiltration will increase the potential for groundwater recharge, which has been significantly depleted. By using bioretention the water output of the system is at a lower temperature than with pond technologies and there are significant water quality benefits.

Summary of measures to be applied at the Sterrett School site

	Function or benefit						
Feature	Recharge of ground water	Reuse of captured runoff	Disconnection of storm drainage from sewers	Reduction in storm runoff volume	Improvment in runoff quality	Watershed quality	Habitat creation and maintenance
Separate sewers				x			
Cul-de-sac creation			x	x	x		x
Regrading	x		x				
Greenhouse		x	x	x	x	x	x
Cisterns	x	x	x	x	x	x	x
Bioretention	x		x	x	x	x	x
Bioswale	x		x	x	x	x	x
Habitat landscaping	x			x	x	x	x
Restore channel	x		x	x	x	x	
Sheet flow	x		x	x	x		x
Bioretention cells	x		x	x	x		x
Playground surfacing	x			x	x		x
Planting	x			x	x		x
Pervious parking	x			x	x		x
Green islands	x			x	x		x
Porous paving	x			x	x		x
Afforestation	x			x	x		x
Art Creek mosaic						x	x
Brochures						x	x
Watershed curriculum						x	x
Tours						x	x
Paint storm drain inlets						x	x
Demonstration project						x	

Regent Square Gateway Narrative Summary

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Following is the main written material handed in by the team at the conclusion of the charrette, lightly edited and reformatted for consistency with other sections of this appendix.

Statement of Design Problem

The "Gateway" site is at the low end of the urban portion of the Nine Mile Run watershed. A densely developed residential neighborhood (a 60-70 ace catchment within the much larger watershed) contributes to overland flow of stormwater runoff at the site. Storm volumes of 2.5 inches in a 2-year, 24-hour storm (estimated to peak at 1.2 inch per hour) result in overland flow down Old Braddock Avenue of perhaps as much as 50-60 cubic feet per second. These flows have caused significant erosion where they tumble into Nine Mile Run, just where the main culvert from the upper watershed first opens up into Frick Park. Runoff from the watershed also carries pollutants into Nine Mile Run.

The project site encompasses a number of design issues including:

- Stormwater/stream culvert outlet for a large portion of the watershed;
- Transition from piped to open channel flow;
- Erosion of the natural stream channel;
- Water quality;
- Recreation/educational opportunities;
- Adaptive reuse of underutilized commercial building and drainage facilities;
- Reuse of local "waste" materials where possible (such as slag for constructed soil);
- Pedestrian/vehicle circulation from Braddock Avenue and from residential neighborhoods;
- Upstream trail connections;
- Trail/park gateway design;
- Uphill urban watershed contributing overland flow to project site.

Gateway Site Context in the Nine Mile Run Watershed

The Regent Square Gateway site is in the lower portion of the Nine Mile Run watershed. A watershed master plan has not been developed, however it is reasonable to assume that stormwater detention should not be encouraged in the lower portions of the watershed. During a storm event, rainfall in the upper portions of the watershed collects and drains along streets and through storm sewers and eventually makes its way to the outfall at the east end of Frick Park. Stormwater that is generated in the lower portions of the basin such as the Gateway study area should be conveyed to Nine Mile Run as soon as possible so that it does not combine with the peak flows from the upper portions of the basin; in other words, the goal is not to detain stormwater in this area.

While stormwater detention should not be encouraged, filtering of stormwater, especially the "first flush" of stormwater generated in small storm events, should be effected to prevent sediment, trash, and other pollutants from entering the stream. This can be accomplished in selected areas by constructing small retention ponds (500-3,000 square feet) to intercept storm flows prior to entering into the collection system and being conveyed to the stream.

The emphasis in this portion of the Nine Mile Run watershed is then on enhancement of water quality prior to entering the stream rather than detention to reduce peak flows in the collection system. Additionally, percolation into the groundwater is a resulting tangible benefit that will help to increase base stream flows in Nine Mile Creek.

Trail Connections

The Regent Square Gateway project is the gateway to the eastern end of the Nine Mile Run trail and the connection to Frick Park. There is also an excellent opportunity to connect further to the east to the Edgewood Crossroads project site. The overland trail connection from Gateway to Edgewood is very urban in nature, with five at-grade road crossings and one bicycle/pedestrian under crossing. In addition to making a trail connection with the Edgewood site, significant portions of adjacent neighborhoods can more easily access the Frick Park trail and take advantage of the recreation opportunities.

Regent Square Gateway Catchment

The Regent Square Gateway catchment is an assumed "watershed" that encompasses natural topography and the stormwater collection system. The estimated boundaries of this catchment were developed based on the knowledge of the existing stormwater sewer systems upslope of the site. It was assumed that all of the known inlets adequately capture the runoff from

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the storms with the exception of the inlets located on South Braddock Avenue. Stormwater bypasses these inlets due to the steep slopes on the road. This condition has been observed during storm events. Stormwater flowing along Braddock Avenue drains directly to Old Braddock Avenue just below the former Foodland upper parking lot entrance. The catchment contains approximately 64 acres. The peak flow for a two year storm is approximately 60 cfs.

The asterisks (*) located on the GIS map of the catchment (in the main report) indicate places where infiltration basins could be located. These basins are designed to filter the stormwater runoff that occurs during low intensity storms and the runoff that occurs during the first flush of higher intensity storms. These systems should be located where there is adequate open space and appropriate soil for infiltration, and should include a reasonable amount of catchment area. The infiltration basins are not designed to detain significant amounts of storm runoff, just to provide a measure of scrubbing or clean-up during small storms and incidental runoff. Suitable locations for infiltration basins are generally within the permeable soil areas noted on maps of local soil/geologic conditions.

Design Issues And Approaches

Terraced Underdrained Basins (TUBs)

Currently most of the watershed drains through the "Foodland" site. No water quality enhancement is being performed on this runoff. The proposed terraced underdrained basins are designed to act as a water quality control to filter the first flush of the storm. Water enters the site off of Braddock Avenue. Instead of the current condition of flowing down Old Braddock Avenue, the runoff will flow through the terraces, located in the underutilized strip of land between Old Braddock Avenue and the Parkway on-ramp. These terraces will be formed from local slag aggregate which will act as a filter for water quality measures. The aggregate will be overlain with soil to allow for planting. The terrace steps will be constructed of clean fill. This will allow for a good foundation for the pathways that will be located on top of them. A perforated underdrain will be located under the terraces and will discharge directly to the channel. Low flow runoff will infiltrate the system and be filtered. The higher flows will flow over the terraces. The terraces will reduce the velocity of the runoff before it enters the overflow spillway. The overflow spillway starts at the end of the terraces, runs along the side of the stream past the headwall (the outfall) of the main Nine Mile Run culvert, and enters the stream via a side channel spillway. The spillway will consist of a concrete lined bottom and the existing side wall of the stream channel. A concrete block at the end of the side channel spillway will be erected to stop the forward motion of the water. The stormwater will then build up in the channel until it spills over the side wall. This will decrease the hydraulic energy in the stream and lower the erosion potential of the stream.

Energy Dissipation in Channel

Adjacent to the "Trailhead/East Park Entrance" is the Nine Mile Run culvert outfall. This is the place where Nine Mile Run emerges after being culverted for at least three miles. Also emerging to the open air here is a 60 inch storm drain and a 54 inch storm drain which together provide drainage for a large portion of the upstream watershed. Also emerging at the site is a 24 inch pipe that drains a portion of the Regent Square neighborhood directly above the site. During heavy storm conditions, this site, where all of the outfalls converge, provides a wonderful opportunity to view the power of nature through the medium of water.

The design of this area addresses the need for erosion control, dissipation of water flow energy, recreational access to water during low flow periods and an overall celebratory approach to provide people with safe views of severe water conditions during storm events.

Water will move through the area in various configurations depending upon weather conditions. During base flow periods there will be a small flow from the Nine Mile Run culvert and other discharging pipes. This will increase over the years as upstream area aquifers become recharged. Seepage from the terraced infiltration system will trickle along the underdrain channel, flow through the culvert headwall and drip onto the streambed below. During high flow periods a side channel spillway will convey runoff from the terraced basins into the channel from its south side.

A series of sculptural elements are proposed for the stream channel itself that refer visually to fluid forms and will interrupt and manipulate flows during high water periods. These fluid forms will dissipate energy and will create dynamic water conditions just below the outfall. Additional stone and concrete elements will be located along stream banks to further mitigate erosion.

Downstream Erosion Protection—Methods & Materials

There is a large amount of erosion downstream of the culvert outfall. In order to protect the proposed trails in Frick Park, the erosion, especially the side bank erosion, should be controlled. Erosion control measures should range from conventional structural methods to bioengineering methods.

Parking Lot Filter Bank

Stormwater runoff from the upper parking lot will flow across the asphalt, onto grass filter strips. The filter strips will perform a low level of water quality enhancement for the runoff. The excess runoff enters inlets, located within the filter strips. The inlets will contain trash racks to filter the larger objects. The water is piped from the inlets into constructed soil located next to the existing retaining wall. The soil is composed of slag aggregate, with a topsoil top to allow for vegetation and trees to be planted. Perforated pipes are located in the topsoil area. Stormwater percolates though the pipes and saturates the top-

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soil. Once the topsoil is saturated, the remaining runoff filters through the slag aggregate and is collected by an underdrain.

Planting on Site

Planting can accomplish many functions in the watershed. Foliage on trees and shrubs can intercept a portion of rainfall so that it doesn't fall directly on pavement and pick up additional pollutants. Trees, shrubs and grasses also absorb water through their roots, hold water and transpire it through their leaves, use carbon dioxide and create oxygen. We recommend hardy, non-invasive species of trees and shrubs, and low-maintenance native prairie grasses.

Gateway Trail Entrance

There has been a recognized need for a second entrance to Frick Park since 1932, though the need has never been realized. We propose this point—already an "unofficial" entry for joggers and walkers—as an obvious gateway to the east side of the park. As a trail head, it is located adjacent to where Nine Mile Run emerges from a culvert that encloses the creek for three miles upstream. The entrance is designed to accommodate hikers, joggers, bicyclists and maintenance vehicles. As an understated gathering place at the trail head, it provides wayfinding information, trash disposal and restroom facilities in the adjacent building. The remnants of an old set of trolley tracks mysteriously disappear here under an embankment, marking a former era of the site. As a viewing area of the emerging stream, tranquil during low flow but a dramatic event during heavy rainfall, this site also offers a place to celebrate and learn from the stream.

Traffic Mitigation—Pedestrian, Bicycle & Automobile Access/Circulation

Traffic on Braddock Avenue is a major design constraint for easy access to the building space and the trail gateway. One of the goals of the project is to provide safer and better opportunities for pedestrians, bicyclists and motorists to access the site. A traffic signal at the entrance to the site will provide a safe route for neighborhoods to the east to walk or bicycle to the site. Pedestrians from the Regent Square neighborhood on the hill above the site would benefit from stairway access to the trail, probably downstream of the outlet structure. There is potential for a trail connection to the Transit Parkway and Edgewood Crossroads site. This connection would open up several more neighborhoods and trail access points for the Nine Mile Run Trail.

Adaptive Reuse of Existing Building (Program Opportunities)

The site contains a currently underutilized building, the upper 16,000 square foot level of which was once a Foodland grocery store. The building also has two lower partial floors, some of which are currently occupied by various businesses. The large upper story is under consideration for redevelopment as another single occupant retail store. The parking layout depicted to the north of the building would accommodate that function.

We recommend that the lower levels of the building be adapted to service users of the park, possibly including the following options. The local municipalities' parks & recreation departments could use the space to offer watershed/ecological education, research space for hydrologists, biologists and ecologists, and public recreation facilities. In addition, private retailers could offer bicycle rentals and food services.

Further Considerations and Opportunities

Fixing Culvert Leaks

Approximately two hundred feet before the culvert daylights at the site, there is a hole located within the culvert. All of the base flow and some of the stormwater runoff that flows through the culvert disappears in this hole. The water falls into either an abandoned sewer pipe or mine. The hole was patched, but approximately 15 feet upstream, a new hole developed. A permanent solution to the disappearing water as well as determining where the water goes, is required prior to the development of this site.

Retention at Highway Interchange

The Interstate 376 Interchange, located to the east of the site, contains an open space. This open space is a possible location for a retention pond. A geological study needs to be performed for the site to determine the feasibility for a retention system there.

Residential Programs

The following is a list of alternative techniques to deal with wastewater at the residential level. These measures could either be included in a pamphlet delivered to homeowners, or they could be promoted in some other way.

- low flow fixtures
- dry wells and infiltration trenches
- rain barrels for garden watering
- rain-filled flex hose sprinkler system

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Policy Team Results

Charrette policy team:

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Following is the main written material handed in by the team at the conclusion of the charrette, lightly edited and reformatted for consistency with other sections of this appendix.

Goal, Objectives, and Conclusions

The policy team recommends the following goals and objectives for managing the infrastructure and improving the human and natural environment of the Nine Mile Run watershed. These statements reflect our understanding of the purposes of the charrette, the possibilities illustrated by the design teams, and the long-term desires of the people of the watershed.

Goal

Restore the Nine Mile Run human and natural ecosystem to sustainable health.

Objectives

1. Establish coordinated planning, maintenance, and management of infrastructure to improve efficiency, reduce costs, and achieve "lighter, greener, cheaper, smarter" infrastructure.
2. Restore the watershed to a more natural hydrology and healthy aquatic and riparian ecosystems.
3. Enable community economic development and revitalization in a manner that is consistent with Objectives 1 and 2.

Conclusions

The charrette has considered a range of stormwater management options for the Nine Mile Run watershed. We have identified essential steps that may be taken in order to address current problems and issues facing the communities and municipalities within the Nine Mile Run watershed.

A vision of improved human and ecosystem health for the future begins with change in the present. A renewed stormwater and sewer infrastructure integrated into the neighborhoods and landscape of the watershed is needed. This infrastructure must use natural ecological processes to maximum advantage. In this way we can improve the infrastructure and simultaneously beautify the city and improve social and economic conditions. The ultimate goal is to restore the Nine Mile Run human and natural ecosystem to sustainable health.

Any initiative of this size requires ambitious long term objectives and realistic, affordable short term steps. Substantial thinking and planning on the part of the citizens and elected representatives of the watershed is a prerequisite to achieving the goal and objectives.

A key step in successful watershed management will be the establishment of a coordinating body authorized to pursue the objectives. Planning and implementing the recommended systems requires an entity with the authority, long term purview, and fundraising ability to coordinate and maintain actions throughout the watershed.

The first objective must be the elimination of existing human health hazards. This will be achieved through the eventual removal of all sanitary sewer overflows and management and reduction of combined sewer overflows. Investment in the present will produce a multi-purpose and cost-effective infrastructure in the future. Current short term fixes which require large expenditures and result in flawed systems must be replaced by long term thinking and solutions based on lighter, greener, cheaper, safer engineering, leading to reduced spending on the infrastructure over time.

An equally important objective is the restoration of the hydrological and biological systems to health. This means restoring the stream, wetland, and upland habitat of the watershed as well as eliminating the additional pollutants now flushing during storms from roads, parking lots, and yards into Nine Mile Run. Part of the plan to obtain this objective includes understanding, revealing, and utilizing the natural systems of the watershed. Monitoring water quality and stream biota over time will be important to measuring our success, because Nine Mile

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Run integrates all that is happening in the watershed, good and bad.

A third objective is to devise long term economic development strategies, cognizant of current economic realities, which will attract new forms of employment to the region. These new employers would be attracted by the region's visual, recreational, and ecological amenities. Healthy environments will invite environmentally sound residential and commercial redevelopment. The involved communities can develop many reinforcing linkages between economic and social development and the implementation of more effective stormwater and sewer infrastructure.

The result of innovative changes like the ones described above is the creation of natural amenities which are at the doorstep of all citizens. These amenities are treasures to the area because they function to sustain the regional watershed and allow the community to reap the health and economic benefits of such changes now, and long into the future.

The charrette policy team has developed general recommendations and suggested action items for cooperative management which focus on infrastructure, ecology, and community development. Separate groups within the policy team addressed each of these three objective areas. Their reports follow.

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Agencies, Programs, And Terminology In Watershed Restoration And Community Redevelopment

The following acronyms and names are used in the presentations of the policy teams results in the following pages.

ACT 167	Pennsylvania Stormwater Management Act (1978)
AIA	American Institute of Architects
ALCOSAN	Allegheny County Sanitary Authority
Allegheny Conference	Allegheny Conference on Community Development, a regional economic development organization comprised of Pittsburgh's major industry leaders.
APWA	American Public Works Association
ASCE	American Society of Civil Engineers
ASLA	American Society of Landscape Architects
BAMP	Builders' Association of Metropolitan Pittsburgh
CDBG	Community Development Block Grant (U.S. Department of Housing and Urban Development)
DCED	Pennsylvania Department of Community and Economic Development
DEP	Pennsylvania Department of Environmental Protection
EPA	U.S. Environmental Protection Agency
FEMA	Federal Emergency Management Agency
GSPIA	Graduate School of Public and International Affairs, University of Pittsburgh
Heinz School	Heinz School of Public Polity and Management, Carnegie Mellon University
NGO	Non-Governmental Organization
NPDES	National Pollution Discharge Elimination System
NRCS	U.S. Natural Resources Conservation Service
Pennvest	Pennsylvania Infrastructure Investment Authority. Provides grants and low-interest loans to communities for improvements to drinking water, sewage treatment and stormwater management facilities.
PRA	Pittsburgh Regional Alliance. Coordinates economic development efforts among six regional organizations: the Greater Pittsburgh Chamber of Commerce, Penn's Southwest, the Regional Industrial Development Corporation, the World Trade Center, the Pittsburgh High Technology Council and the Southwestern Pennsylvania Industrial Resource Center.
PPA	Pennsylvania Planning Association
QAQC	Quality Assurance/Quality Control
SOP	Standard Operating Procedure
TMDL	Total Maximum Daily Load. TMDLs are site specific water quality discharge criteria required by the Clean Water Act for bodies of water that are failing to become "fishable, swimmable" even with the application of effluent standards.
TRWWDP	Three Rivers Wet Weather Demonstration Program
USGS	U.S. Geological Survey
WA	Watershed Authority (proposed in this report; also referred to as the "Watershed Management Entity")

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Infrastructure Group Summary

Watershed Management Entity

The centralized management of stormwater infrastructure would prove most efficient. Some form of authority, utility, or district is needed to address the infrastructure and ecosystem issues that occur throughout the watershed, across municipal boundaries. Here, we use the term authority. The affected communities will have to establish the proper legal form and powers of this watershed management entity.

A watershed authority would not only reduce cost, it would maximize benefits to citizens and the ecosystem as it corrects inadequate infrastructure. A steward to the ecosystem, an authority would also interact with other levels of government, opening the lines of communication between the different players, while providing maximum support of the infrastructure. The ultimate plan is to meet the ecological goals, both short- and long-term.

Elected officials will work with a citizen's board to develop and implement action plans to achieve restoration goals for the watershed and its people. A general manager will oversee the authority's efforts. The representative group will approve plans for infrastructure construction and reconstruction submitted to the authority. They will set base rates for stormwater infrastructure user fees, publish codes and/or evaluate and recommend modifications to municipal and county codes, and describe the relevant devices, such as Best Management Practices (BMPs), for parcel development and redevelopment. The board will also oversee design of an incentive plan based on cost reductions from increased infiltration of stormwater. The authority will be staffed and funded to effectively manage the system, and will have the authority to contract out necessary work.

During the first and second year, the watershed authority will act as a model of inter-municipal watershed cooperation. After a couple of years, the authority will become a more formal entity enabled by legislation. In the long term, 50 years and more, the ecological goals will be fulfilled.

Infrastructure

Sewer and stormwater infrastructure includes not just catch basins and pipes and treatment plants. It should also take advantage of the natural capacities of soil and vegetation to absorb water and treat urban runoff pollutants. We call this larger conception "green infrastructure." The charrette's design teams have illustrated many of the strategies and techniques that could be applied. Green infrastructure can be encouraged and implemented at several levels: the watershed, the community, and individual properties.

At the watershed scale, the objectives include reducing Combined Sewer Overflows (CSOs) and stopping Sanitary Sewer Overflows (SSOs). All players, including citizens and elected officials, take part in achieving this goal. Leadership from the watershed authority will assist in the reduction of CSOs and the abatement of SSOs. These short-term objectives must be achieved before the long-term goals can be tackled. The long-term objective at the watershed scale is the green engineering of the whole system, leading to restoration of the watershed's natural and human ecosystem. To support achievement of this goal, the authority would implement "regional" (larger scale) retention and recharge projects where appropriate, and assist in implementation of green infrastructure at the community and property levels.

The objectives at the community scale are to implement green infrastructure on city streets, parks and other public properties, and to encourage actions on private properties. It is important that the responses for different communities be calibrated to the varying needs, desires, and capacities of the different communities. At the community scale, the municipalities, with some assistance from the authority, will lead the implementation of green infrastructure measures on their streets, parks, schools, and other public properties. These measures will improve civic landscapes, recreation facilities, wildlife habitat, and other amenities. It is important that the municipalities and the watershed authority educate citizens and property owners about the techniques and benefits of green infrastructure. The time frame for the implementation of these tasks varies from medium- to long-term.

At the individual parcel scale, maximum infiltration of runoff to the soil and minimum contribution to the CSO and SSO problems is crucial. As implementation of green infrastructure spreads to larger numbers of properties, the cumulative value of the green infrastructure will increase. The individual parcel owners are responsible at this level of implementation. They could be incentivized through infrastructure user fees, with fee reductions for high rates of on-site stormwater infiltration and treatment. At this scale, the time frame is continuous and varies from medium- to long-term.

The following page summarizes the above discussions of the watershed authority and the management of the infrastructure.

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Infrastructure Action Plan

I. Establish a Coordinating Body			
Action Steps	**Performance Objectives**	**Who**	**Time Frame**
1. Communities form the co-ordinating body, authorize it to obtain funding, manage infra-structure, manage the stream system, hire staff, contract out work. 2. Approve plans for infrastructure construction and reconstruction submitted by municipalities. 3. Set base rates for use of storm-water infrastructure, publish codes, describe recommended practices for redevelopment. 4. Design incentive plan based on cost reductions for increased infiltration (reduced runoff).	Efficient central management Protect citizens Reduce costs Maximize benefits to citizens and ecosystem Steward the ecosystem Interact effectively with other levels of government Meet human health and ecosystem goals!	Elected municipal officials, public board of the WA General manager, staff, and contractors of the WA	1-2 years: start out to be a model of watershed co-operation 2 years+: lead to more formal utility/authority entity enabled by legislation Near to long-term: meet infrastructure, human health, and ecosystem goals/objectives

II. Manage the Infrastructure			
Action Steps	**Performance Objectives**	**Who**	**Time Frame**
Watershed Scale: 1. Lead SSO/CSO solutions. 2. Implement regional retention and recharge, coordinated with recreation, where feasible. 3. Encourage infiltration at community and property scales.	Reduce CSOs Stop SSOs Green engineer the watershed's infrastructure (including natural systems)	WA, with municipalities	Short to long-term
Community Scale: 1. Implement infiltration in streets, parks & schools. 2. Improve habitat. 3. Improve recreational amenities.	Reduce CSOs Stop SSOs Implement "green streets" Calibrated responses for different community types	Municipalities with assistance from WA	Medium to long-term
Individual Parcel Scale: 1. Establish user fees & incentives for infiltration.	Maximum infiltration, minimum contribution to sewer overflows	Parcel owners	Medium to long-term

Natural and Human Ecology Group

Summary

The objectives of the Ecology Group's recommended action plan recognize both the immediate ecological monitoring needs and the long-term vision of sustainable urban landscape—a landscape that may take as many as 100 years to emerge. These include restoration of the hydrologic cycle and biological integrity of the stream in a way that produces a quality human environment.

The basis of any deep understanding of the Nine Mile Watershed must begin with inventory and assessment. We have attempted to outline the immediate and long-term monitoring programs necessary to provide sound environmental input for the redevelopment and planning process.

Redevelopment is a long-term process with many immediate focal areas. We have outlined priorities for protection and preservation of watershed lands, keeping ecosystem health and function as the framework for our recommendations.

The results of the monitoring and assessment process will be communicated to community residents, municipalities, and the watershed authority. Based on recommendations from this communication loop, monitoring goals, environmental priorities, and attainable conditions will be modified to ensure and measure success of watershed restoration programs on human health and the environment.

The action plan of the Ecology Group is designed to:

- Provide the community and watershed authority an understanding of the existing natural and human ecological conditions.
- Involve the community in the environmental inventory. We also suggest that a percentage of all research and capital dollars be simultaneously invested in revitalizing watershed communities in an equitable manner.
- Develop inventory, analysis and modeling tools that make it possible to both understand and manage the watershed now and in the future. The system must be able to evolve with technology and changing community goals.
- Provide an ecological basis to identify critical areas and use this information to set priorities and develop attainable goals and redevelopment criteria.
- Define the existing baseline ecological conditions as a measure for assessment of the watershed ecological function and how well the watershed restoration actions are working.

The tasks of the action plan, detailed in the following pages, are:

I. Restore the hydrologic regime.
II. Improve the stream channel and flood plain.
III. Conserve urban watershed habitat.
IV. Monitor water quality and biology.
V. Integrate and acquire feedback.

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I. Restore the Hydrologic Regime

Action Steps	Performance Objectives	Who	Time Frame	Funding Sources	Potential Challenges
1. Implement a long-term basin-wide hydrologic monitoring program.	Network of gauges and monitoring stations operating in best locations. QAQC and SOPs	USGS and WA	1 year and continuing	WA, Boroughs, USGS	Lack of coordination between agencies or lack of regulatory driver
2. Develop a hydrologic and hydraulic watershed model.	Calibrate to existing condition, then to level necessary to detect a significant flow change	WA and consultants	2 years	WA, Boroughs, EPA	Lack of coordination between agencies or lack of regulatory driver
3. Update flood plain maps.	Use FEMA-approved models to update maps to existing conditions; extend upstream into culverted areas	FEMA and WA	3 years	WA, Boroughs, EPA	FEMA regulations not geared to closed urbanized systems No requirement to update
4. Identify flood prone structures and areas and develop strategies to address them.	Quantify location and extent of flooding	Municipalities and WA	1 year after flood plain study	WA, Boroughs, FEMA	FEMA regulations not geared to closed urbanized systems No requirement to update
5. Establish attainable redevelopment criteria for stormwater hydrology.	Develop criteria for duration, flow rate, timing, volume and velocity	WA	Immediate and update in three years	WA	Lack of existing regulatory form Consistency and integration with economic development Lack of incentives
6. Perform watershed infiltration, detention, retention opportunities analysis.	Complete map of opportunity areas and priorities based on soils, geology, conservation areas and economic development	WA, urban redevelopment authorities, municipalities, community development corporations	1 year	EPA, State, watershed organizations and foundations	None

II. Improve the Stream Channel and Flood Plain

Action Steps	Performance Objectives	Who	Time Frame	Funding Sources	Potential Challenges
1. Inventory stream channel and flood plain geomorphology.	Collect material necessary for geomorphic analysis; monitor over time Prepare conceptual plan QAQC and SOPs	WA, municipalities, and consultants	1 year, and following flow modification and restoration projects	WA, Boroughs, EPA, NRCS, Army Corps of Engineers	Lack of coordination between agencies or lack of regulatory driver
2. Perform geomorphic analysis and sediment transport modeling.	Determine critical velocity, bank full flow, channel-forming flow, 10 yr and 100 yr water surface elevations & flows, watershed erosion rates, stream bank and channel stability	WA, municipalities, and consultants	2 years	WA, Boroughs, EPA, NRCS, Army Corps of Engineers	Lack of coordination between agencies or lack of regulatory driver
3. Identify existing and future channel stability problems.	Locate, describe and prioritize restoration sites	WA, municipalities, and consultants	2 years	WA, Boroughs, EPA	Lack of coordination between agencies or lack of regulatory driver
4. Fix critical stream bank erosion problems.	Stabilize critical areas Ensure public safety	WA and consultants	3 years	WA, Boroughs, EPA	Lack of coordination between agencies or lack of regulatory driver
5. Create long-term channel restoration plan for new watershed flow regime.	Create channel equilibrium, consistent with targeted flows and bio-engineering standards targeted to ecosystem restoration goals	WA and consultants	3 years	WA, Boroughs, EPA	Lack of coordination between agencies or lack of regulatory driver
6. Develop a buffer policy.	Stabilize channel and reduce non-point source pollutant loads Width based on land use, physical setting and habitat value	WA, County, and consultants	1 year	WA, EPA	Consistency with existing local buffer codes
7. Implement a greenway program.	Promote greenway connectivity, recreation stewardship	WA and consultants	2 years	WA, Boroughs, EPA	Consistency with existing local buffer codes

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III. Conserve Urban Watershed Habitat

Action Steps	Performance Objectives	Who	Time Frame	Funding Sources	Potential Challenges
1. Inventory watershed and establish natural resources GIS database.	Establish tiered baseline data for planning and programming QAQC and SOPs	WA, TRWWDP, and consultants	2 years	WA, County, State, EPA	None
2. Perform critical areas analysis.	Based on integrated, multi-level analysis of restoration potential, redevelopment potential and ecosystem value	WA and consultants	2 years	WA, Boroughs, EPA	None
3. Develop short-term and long-term strategies for land management and protection. (i.e. criteria for development: create linkages between ecosystems, protect critical areas, prioritize enhancement).	Achieve a predetermined level of ecosystem enhancement	WA, County, State, and consultants	3 years	WA, EPA	Lack of coordination between agencies or lack of regulatory driver
4. Implement a land acquisition and easement program.	Achieve physical linkages	WA	4 years	WA	Consistency with existing redevelopment program and Allegheny County Greenway Programs
5. Develop natural resources redevelopment criteria such as open space requirements, banking, and fee in lieu of on-site program.	Increase watershed habitat by 12 percent (suggested goal) Provide ecosystem restoration options that can be realized either on-site or in priority areas Increase connectivity and renew ecosystem function	WA	1 year and revised in 3 years	WA	Consistency with existing redevelopment program and Allegheny County Greenway Programs

IV. Monitor Water Quality and Biology

Action Steps	Performance Objectives	Who	Time Frame	Funding Sources	Potential Challenges
1. Implement a long-term chemical and biological monitoring program which reflects the impact of failing infrastructure.	Network of monitoring stations in best locations Data quality adequate to calibrate the model Biological modeling to establish reference conditions QAQC and SOPs	WA and consultants	1 year	WA, County, State, EPA	Will be required by Clean Water Act (NPDES, TMDLs) Must be consistent with state programs
2. Model existing and future water quality.	Model must be adequate for provided baseline loads and efficacy in predicting load reduction strategies	WA and consultants	3 years	WA, EPA	Will be required by Clean Water Act (NPDES, TMDLs) Must be consistent with state programs
3. Perform attainability and alternatives analyses.	Based on integrated, multi-level analysis of watershed load reduction requirements, efficacy of alternatives, and potential to restore ecosystem value Provide flexibility options	WA and consultants	4 years	WA, EPA	Options must be consistent with County and State programs
4. Revise redevelopment criteria.	Encourage impervious surface reduction, rooftop drain disconnection, and permeable paving Require passive treatment of first-flush runoff from impervious areas	WA	1 year	WA	Lack of coordination between agencies or lack of regulatory driver
5. Implement pollution prevention program.	Set municipal operations standards and commercial and residential maintenance guidelines	WA and consultants	2 years	WA, Boroughs, EPA	Lack of coordination between agencies or lack of regulatory driver

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V. Integrate and Aquire Feedback

Action Steps	Performance Objectives	Who	Time Frame	Funding Sources	Potential Challenges
1. Communicate results to public in an understandable way.	100% accessibility for all studies, reports and documents Simple language Internet access Provide a public information gateway for questions and comments	WA, Municipalities, local NGOs	2 years and update as needed	WA, Boroughs, EPA	None
2. Involve public in monitoring, storm drain stenciling, stream cleanups, naming, adopting, planting, riparian restoration, and roof drain disconnects.	Build consensus and stewardship through experiences in the field and participation in monitoring and strategizing Develop volunteer programs, educational materials and access to laboratory testing for urban streams with likely health impacts	WA, Municipalities, local NGOs	2 years and update as needed	WA	None
3. Develop environmental curricula for schools and watershed.	Enable children's sense of ownership, responsibility and voice in issues of environmental consequence	WA, Municipalities, Board of Education	2 years	WA	Must be consistent with County and State school education program requirements.
4. Coordinate existing programs within the watershed.	Require information exchange Analyze and rank the results of actions, for multiple benefits and community understanding Each stakeholder addresses the results WA facilitates, and provides QAQC	WA, Municipalities, local NGOs	Continuous and ongoing	WA	None

Community Development Group Summary

This subgroup of the policy team examined ways community development* policies, programs, and activities can bolster efforts to rehabilitate the sewer and stormwater infrastructure and restore the ecosystem. Many beneficial linkages between development activities and infrastructure/ecosystem improvements are available.

The following principles can guide creation and elaboration of these linkages:

- Development should be cognizant of watershed issues and impacts, especially cumulative impacts.
- Coordinate the planning of development that has multi-municipal impacts to stormwater and sewage infrastructure, ecosystems, traffic, zoning, land use, citizen input & outreach.
- Assure development occurs as planned and agreed upon to enhance and fit with existing and pending development.
- Develop coordination and oversight processes to effectively accommodate unplanned private initiative and new opportunities.

The following action plan will enable the communities to take advantage of the mutually reinforcing linkages between improved infrastructure and desirable development:

I. Reconcile zoning and land use ordinances of the watershed communities—Pittsburgh, Wilkinsburg, Swissvale, and Edgewood—for compatibility with watershed objectives and goals.
II. Review local codes (e.g. building, street design, drainage, plumbing, property maintenance) and procedures for consistency with watershed objectives and green infrastructure measures.
III. Identify and correct existing development constraints (legal, physical, and financial) caused by inadequate infrastructure.
IV. Develop a coordinating mechanism for municipal development plans, to assure projects do not contradict each other or overall ecosystem/development objectives.
V. Enhance the existing stormwater management program (Act 167 plan) to reflect watershed needs, enable green infrastructure, and facilitate community, social, and economic benefits.
VI. Educate citizens, officials, and developers that good development and a healthy environment are compatible and reinforcing.

The following pages present an initial elaboration of this action plan. Further below, the team presents some comments and recommendations on local codes and policies, and an initial list of resources and strategies that could assist communities in developing and implementing the suggested actions.

*Development includes all redevelopment activities as well as new development on unbuilt-upon land.

R0022309

Community Development Action Plan

I. Reconcile Zoning and Land Use Ordinances of the Watershed Communities for Compatibility with Watershed Objectives and Goals					
Action Steps	**Who**	**Time Frame**	**Estimated Costs Funding Sources**	**Outreach and Education**	**Potential Challenges**
1. Establish technical review panel. 2. Convene planning commissions to approve reconciliations. 3. Elected bodies adopt recommendations.	Municipal planning commissions and staff, governing bodies, WA DCED could facilitate Resources: PPA	12 Months	$20,000 Municipalities, DCED, State Planning Assistance Grants, County	News media, public meetings, public hearings	Turf issues Justification Lack of County Planning Department

II. Review Local Codes and Procedures for Consistency with Watershed Objectives and Green Infrastructure Measures					
Action Steps	**Who**	**Time Frame**	**Estimated Costs Funding Sources**	**Outreach and Education**	**Potential Challenges**
1. Establish technical review panel. 2. Make recommendations to governing bodies. 3. Adopt recommendations.	Municipal engineers, attorneys, governing bodies, WA Resources: APWA, AIA, ASCE, ASLA, PPA, Brandywine Conservancy	12 months	$40,000 Municipalities, DCED	News media, public meetings, public hearings, BAMP	Reactions of builders and developers Overcoming implementation inertia (e.g. building inspectors)

III. Identify and Correct Existing Development Constraints (legal, physical, and financial) Caused by Inadequate Infrastructure

Action Steps	Who	Time Frame	Estimated Costs Funding Sources	Outreach and Education	Potential Challenges
1. Identify economic development problems related to infrastructure. 2. Prepare inventory and evaluation of existing infrastructure systems.	Municipal Engineers, Departments of Public Works, Community Development Depts/Economic Development Authorities, Governing bodies, ALCOSAN, WA Resources: Pennvest, CDBG State Revolving Fund, DCED, Heinz School, GSPIA, Chamber of Commerce	12 Months	$100,000+	Early citizen notice, public processes	Inter-municipal cooperation Obtaining seed money
3. Develop corrective action plan. 4. Develop capital budget.		24 Months	$100,000+		
5. Implementation of capital budget.		72 Months	$1,000,000+		

IV. Develop a coordinating mechanism for municipal development/ redevelopment plans, to assure projects do not contradict each other or overall ecosystem/development objectives					
Action Steps	**Who**	**Time Frame**	**Estimated Costs Funding Sources**	**Outreach and Education**	**Potential Challenges**
1. Obtain and review current plans.	Municipal planning commissions, governing bodies, WA Resources: DCED, Allegheny Conference, Pittsburgh Regional Alliance, Universities	12 Months	Small Amount	Local officials, business community, public, entrepreneurs	Turf issues Who convenes? Differing development priorities
2. Establish periodic joint planning commission meetings to negotiate inter-municipal issues. 3. Governing bodies incorporate items into development plans, and if desired, establish intergovernmental cooperation agreements.		Ongoing			

V. Enhance the existing stormwater management program (Act 167 plan) to reflect watershed needs, enable green infrastructure, and facilitate community, social, and economic benefits

Action Steps	Who	Time Frame	Estimated Costs Funding Sources	Outreach and Education	Potential Challenges
1. Identify existing plan shortcomings and watershed needs. 2. Update existing ordinances & regulations.	Municipal officials and governing bodies, County, WA Resources: TRWWDP	12 months	$25,000	Public, municipal officials, state legislators, DEP, economic development agencies, property owners	Politics at municipal, county, and state levels Reactions of developers
3. Amend Act 167.		10 years	Unknown		

VI. Educate citizens, officials, and developers that good development and a healthy environment are compatible and reinforcing

Action Steps	Who	Time Frame	Estimated Costs Funding Sources	Outreach and Education	Potential Challenges
1. Identify communication media.	Municipalities, WA, environmental groups, Chambers of Commerce, ALCOSAN, County Health Dept., WQED, League of Women Voters, public relations firms Heinz School, Environmental City Initiative, Schools, Hosanna House, Redevelopment agencies, TRWWDP, Green Building Alliance	6 months	$60,000	Public officials, citizens, developers Media to include: TV, radio, public service announcements, public schools, stenciling program, workshops, newspapers	Apathy Obtaining money Politics
2. Develop and implement a marketing strategy.		6 months	$100,000		
3. Develop and implement integrated demonstration results.		2 years	$200,000		
4. "Export" results of demonstration projects, and continue outreach.		Ongoing	Some		

Land Use Ordinances and Other Regulatory Recommendations

In order to implement short and long-term watershed restoration and stormwater management objectives and projects, a number of changes are needed to existing municipal, county, and state regulatory requirements. Some specific recommendations developed in the limited time of the charrette are listed below. Note that several of these recommendations address items I and II in the community development action plan presented above.

Establish a long-term, sustainably funded, multi-municipal, watershed-wide environmental management entity.

To ensure that the goals and objectives of maintaining and preserving human health and ecological integrity are met throughout the watershed, it is necessary to establish a watershed-wide entity with the authority to work across municipal boundaries.

1. Immediately after the charrette, participants will send a joint letter to the Pennsylvania DEP Southwest Regional Director requesting a meeting to discuss the pending DEP Order requiring municipalities to comply with the Sewage Facilities Act and the Clean Streams Law. We should recommend that DEP use this opportunity to broaden the requirements for municipalities subject to the order to undertake cooperatively a broader range of watershed management activities, including forming a management authority.
2. Create a long-term, sustainably funded, multi-municipal, watershed-wide environmental management entity.
3. Create a stormwater banking system within the watershed using the East Hills Shopping Center and similar sites to create mitigation opportunities for smaller sites.
4. Provide a mechanism for Wilkinsburg to receive financial incentives to adopt "green zoning."

Municipalities review existing ordinances and revise to incorporate stormwater requirements into existing zoning ordinances.

1. Deconflict requirements for curb cuts, catch basins, and street design with stormwater management considerations.
2. Allow landscaping for water retention as well as aesthetics and other traditional purposes.
3. Existing vegetative cover should be retained as much as possible during development.
4. Stormwater management should be considered in the placing of erosion and sedimentation basins during construction to ensure that erosion & sedimentation and stormwater objectives are adequately met.
5. Minimize impervious surface for new development and reduce or mitigate impervious surface during redevelopment.
6. Require better than one-for-one mitigation of equivalent ground water recharge for impervious surface.
7. Require that parking lot and all other impervious surface designs must include infiltration of stormwater, unless it is proven impossible for the site.
8. Require that redevelopment cannot be approved without compliance with stormwater management/infiltration/impervious surface reduction requirements. Modify Redevelopment Authority policies, as well as land use ordinances.
9. Increase fines for noncompliance to eliminate the economic benefit of noncompliance or establish effective deterrence levels.
10. Examine municipal building codes and county plumbing code to identify barriers to individual stormwater retrofits such as cisterns.

Municipalities create new environmental zoning ordinance provisions.

1. Incorporate Federal Flood Insurance Program provisions in local ordinances to achieve higher ratings for lower flood insurance premiums.
2. Create "official map" for zoning ordinance.
3. Create an "environmental overlay zone" for zoning ordinance.
4. Create policy for acquisition of tax delinquent properties within the overlay zone, resulting in increased green space and stream daylighting.
5. Develop comprehensive environmental sections for zoning and subdivision and land development ordinances.

Improve effectiveness of erosion and sedimentation and stormwater control requirements.

1. Raise enforcement of Allegheny County Conservation District to highest level to improve effectiveness.
2. Allow county health department and conservation district enforcement of Clean Streams Law (requires change in state law).
3. Revise state erosion and sediment control regulations (Chapter 102) to require higher standards.
4. DEP/Allegheny Co. should require changes to local ordinances to incorporate "green" provisions, and give priority for financial grants to communities which adopt "green" zoning.

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Comments on Existing Codes and Ordinances

The policy team spent a short time at the charrette examining local codes and ordinances. There was not time for a comprehensive review. The following selected items indicate some of the barriers that may exist to implementation of the stormwater management techniques illustrated by the charrette's design teams. No doubt there are others barriers in the codes. As indicated by the community development action plan items I and II, local governments should undertake comprehensive reviews of codes and ordinances and reconcile them with watershed and infrastructure restoration objectives, strategies, and techniques.

Municipal Codes and Ordinances

Wilkinsburg's Zoning Code does not refer to the Borough's Stormwater Management Ordinance. It should require compliance with the stormwater ordinance before a permit is issued for building/land use changes, development, or redevelopment.

Wilkinsburg's Stormwater Management Ordinance has a good 'trigger' for stormwater management, i.e. any "development of land which may affect stormwater runoff." However, the ordinance does not address restoration of existing channels, streambed conditions, etc. The ordinance has special standards for the Turtle Creek Watershed; analogous standards, unique to the human and natural ecosystem needs of Nine Mile Run, should be developed and implemented.

Section 7.707 of Edgewood's Planned Commercial Development Ordinance imposes landscaping requirements based on aesthetics. The scope of the section needs to be expanded and provide for greater flexibility to include hydrological objectives.

Section 609.5 of Edgewood's Subdivision Land Ordinance requires connection of storm drains to paved gutters. It should allow use of on-site retention/infiltration systems.

BOCA Codes

The municipalities in the watershed refer to the BOCA codes for building inspection and property maintenance. (BOCA stands for Building Officials Code Administrators.) These codes include provisions which dampen visionary stormwater techniques.

BOCA Property Maintenance Code

Section 303.2 —*All premises shall be graded and maintained to prevent the erosion of soil and to prevent the accumulation of stagnant water thereon, or within any structure located thereon. (Exception: water retention areas and basins approved by the code official.)*

Depending on implementation, this could discourage shallow surface infiltration basins and other techniques that allow or create short-term ponding of water.

Section 303.4—*All premises and exterior property shall be maintained free from weeds or plant growth in excess of 10 inches.... Weeds shall be defined by grasses, annual plants, and vegetation, other than trees or shrubs; provided, however, this term shall not include cultivated flowers and gardens.*

This should be revised so as not to deter grass swales, vegetated drainage channels, native species landscaping (native plants have more robust root systems that increase infiltration, as compared to standard turf grass), etc.

Section 304.7—*Roof water shall not be discharged in a manner that creates a* public nuisance.

Section 508.0—*Drainage of roofs and paved areas, yards and courts, and other open areas on the premises shall not be discharged in a manner that creates a* public nuisance.

Section 302 defines a public nuisance by referring to common law and also as "*any physical condition ... considered an attractive nuisance to children.*" This is a vague definition that could freeze innovation. The communities should carefully define what conditions are actually dangerous or undesirable and disallowed.

Implementation Resources and Approaches

Following is an initial, brainstormed list. No doubt many other resources and strategies exist.

Funding

Resources

- Community Development Block Grants
- Community Facilities Grants
- PennVEST
- Army Corps of Engineers (3 Rivers Wet Weather Demonstration Project)
- Environmental Protection Agency
- TEA-21 (Enhancements & Restoration)
- Foundations
- DCNR - Rivers Conservation Program
- DCNR - Greenways Planning (Key 93)
- Variable Rate Bond Pools (3 Rivers Wet Weather Demonstration Project)
- Developers
- DCED - Economic Development and SPAG
- State Capital Budget
- Water Resources Development Act
- USDA - Farmers Home Administration
- HUD - (Sanders Task Force and Empowerment Zones)
- Tax Increment Financing
- Low Income Relief from Taxation
- Regional Asset District
- Municipal Capital Budgets

Strategies

- Access fundraising technical assistance from some of the following resource agencies:
  - Department of Community and Economic Development
  - Governor's Task Force
  - Army Corps of Engineers
  - Allegheny County Dept. of Economic Development
  - PennVEST
  - Council of Governments
  - Universities
  - 3 Rivers Wet Weather Development Project
  - Department of Conservation and Natural Resources
  - Department of Environmental Protection
- Municipalities work together for funding
- User fees
- In lieu of "fees"
- Mass bidding

Technical/Planning

Resources

- Resources listed under funding plus:
- Natural Resources Conservation Services
- United States Geological Survey
- Allegheny County Health Department
- Pittsburgh High Technology Council
- Environmental Business Network
- Green Building Alliance
- Penns Southwest
- Pennsylvania Economy League
- Pennsylvania Department of Environmental Protection
- Southwest Pennsylvania Commission
- Southwest Pennsylvania Industrial Resource Center
- SPEDD
- Professional Organizations (WEF/APWA/AIA/ASCE/APA/AICP/AWRA)
- Universities - Heinz/GSPIA/Bayer School
- Fish and Boat Commission
- Environmental Organizations - Allegheny Land Trust, Western Pennsylvania Conservancy and others
- ALCOSAN
- Department of City Planning

Strategies

- Ordinances, policies, and regulations
- Curative amendments
- Drainage and development easements
- Eminent domain
- Comprehensive planning
- Tax delinquent properties

Legal

Resources

- Municipal attorneys
- 3 Rivers Wet Weather Demonstration Project
- Pennsylvania Environmental Defense Foundation
- Pennsylvania Environmental Council
- Environmental Law Institute
- University Law Schools (law clinic)
- Pro bono law firm services
- Bar Association
- Department of Environmental Protection
- Legislators

Strategies

- All the strategies listed under technical/planning plus:
- Acquisition
- Legal strategies in support of development
- Legislation
- Litigation
- Incorporation strategies; e.g. land trust

Management

Resources

- Department of Community and Economic Development
- Governor's Task Force
- Allegheny County Department of Economic Development

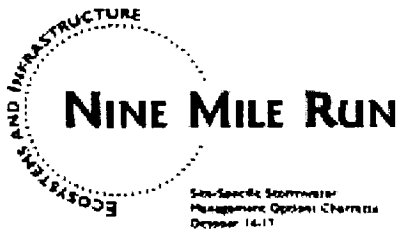
- PennVEST
- Council of Governments
- 3 Rivers Wet Weather Development Project
- Department of Conservation and Natural Resources
- Department of Environmental Protection
- Natural Resources Conservation Service
- Allegheny County Health Department
- Pennsylvania Economy League
- Professional Organizations (WEF/APWA/AIA/ASCE/APA/AICP/AWRA)
- Universities - Heinz/GSPIA/Bayer School
- Environmental Organizations - Allegheny Land Trust, Western Pennsylvania Conservancy and others
- ALCOSAN
- Department of City Planning
- Allegheny League of Municipalities/PSATS/PSBA
- Local Government Academy
- Accounting and business consulting firms
- Community Technical Assistance Center

Strategies

- Joint Municipal Authority
- ALCOSAN
- County Authority
- Environmental Improvement Compact
- Stormwater District (County wide or watershed based)
- Allegheny County department
- Privatization
- Contract operation
- Management Committee/Joint Management Agreements
- Council of Governments
- Nonprofit corporation
- Environmental Advisory Committee

Letter to the Pennsylvania Department of Environmental Protection

The following letter was composed at the charrette, signed by the persons indicated below, and forwarded to the Pennsylvania DEP. The requested meeting took place December 15th, 1998.



October 17, 1998

Mr. Charles Duritsa
Director, SW Regional Office
Department of Environmental Protection

Dear Chuck:

We are participants in a multi-disciplinary planning activity focussing on Nine-Mile Run called the Site-Specific Stormwater Management Options Charrette. We have developed an exciting series of options, recommendations, and design alternatives to address the stormwater, sewage collection, and ecological restoration needs of this degraded watershed. As a result of our work, we have determined that DEP has an immediate and important role to play in implementing these recommendations.

We are aware that the Department has drafted an Order to the Nine-Mile Run communities requiring them to begin an assessment of the watershed sewer system to eliminate violations of the Clean Streams Law. We request the opportunity to discuss with you in the very near future the results of our Charrette and how its findings could relate to your Order and address a broader range of watershed management activities.

Sincerely,

Bruce Ferguson, Charrette Facilitator

Tim Collins, Studio for Creative Inquiry
Carnegie Mellon University

Caren Glotfelty, Goddard Professor of
Forestry and Environmental Resource
Conservation, Penn State

John Stephens, Friends of the Riverfront

Richard Pinkham, Rocky Mountain Institute

John Schombert, 3 Rivers Wet Weather
Demonstration Program

John Childe, Pennsylvania
Environmental Defense Foundation

cc: James Seif, Secretary of Environmental Protection
Terry Fabian, Deputy Secretary for Field Operations
Dr. Hugh Archer, Deputy Secretary for Water Management
Dr. Bruce Dixon, Director, Allegheny County Health Department
Tim Drier, Regional Water Manager

Base Maps for the Four Sites

Each of the design teams at the charrette worked off a line drawing base map of its site, provided at a scale appropriate for trace-overs, measurements, and other design tasks. The detail included streets, sidewalks, buildings, topography, sewers, storm drains, other utilities, and other features. Information for the base maps came from a variety of sources, and site inspections.

Chris Leininger of Sustainable Home Design, Beaver, Pennsylvania, produced these maps.

The base maps are available for inspection at the STUDIO for Creative Inquiry, Carnegie Mellon University, Pittsburgh.

R0022319

United States
Environmental Protection
Agency

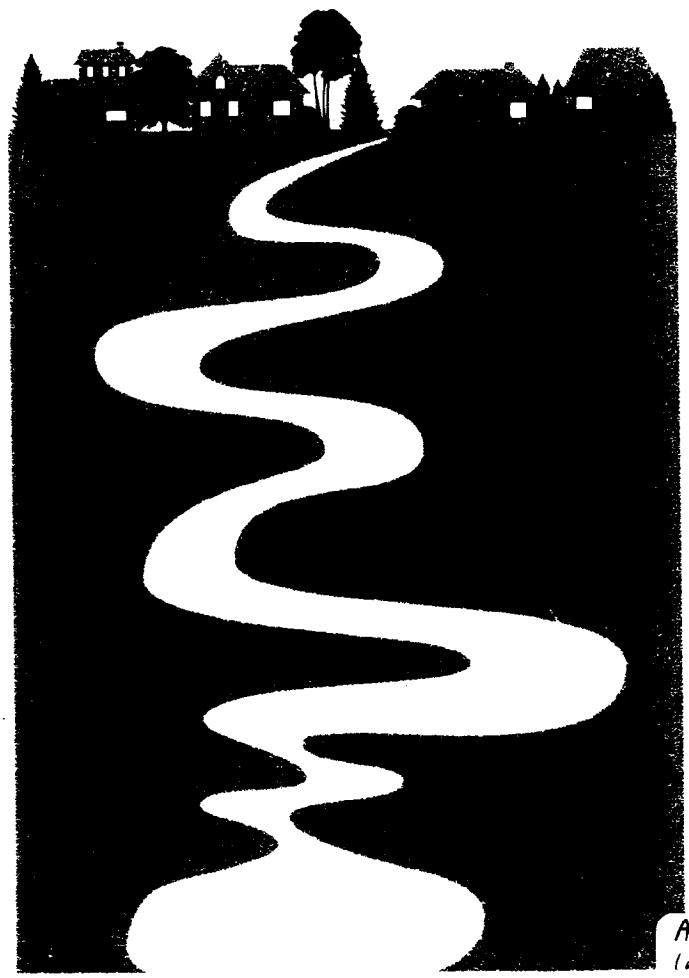
Office of Research and
Development
Washington, DC 20460

EPA/625/R-99/002
July 1999

EPA

National Conference on Retrofit Opportunities for Water Resource Protection in Urban Environments

Proceedings
Chicago, IL
February 9-12, 1998



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EPA/625/R-99/002
July 1999

National Conference on Retrofit Opportunities for Water Resource Protection in Urban Environments

Proceedings
Chicago, IL
February 9-12, 1998

Technology Transfer and Support Division
National Risk Management Research Laboratory
Office of Research and Development
U.S. Environmental Protection Agency
Cincinnati, Ohio 45268

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R0022322

Introduction

Water resource managers have been successful in developing approaches for reducing nonpoint source pollution in newly developing urban areas. Issues become increasingly complex, however, when managers are faced with the challenge of reducing nonpoint source impacts within previously developed urban environments. A diverse assortment of resource management tools, or "retrofits," is being developed, but their implementation has been hampered by a lack of technology transfer opportunities. The ***National Conference on Retrofit Opportunities for Water Resource Protection in Urban Environments*** was designed to address these issues and to transfer much-needed information to state and local water resource practitioners.

Held in Chicago, Illinois, on February 9-12, 1998, the conference program brought together an array of progressive scientists and researchers, along with managers of successful local retrofit projects from across the country. Session topics included retrofit opportunity identification, modeling and monitoring approaches for retrofit applications, conservation design strategies, innovative financing approaches, evaluating results and measuring success, newly emerging technologies, urban revitalization issues, riparian reforestation, and public education and involvement programs.

During the conference, a series of speakers presented papers, 43 of which are reproduced in these proceedings. The purpose of this document is to present these papers and provide information to individuals unable to attend. All papers included were peer reviewed. This document will be useful to individuals who are interested in information about retrofitting techniques and approaches to improving protection of urban water resources. A list of the nearly 300 attendees is provided following the papers.

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Acknowledgements

The success of the conference and the preparation of this document are due largely to the efforts of the presenters as well as the following individuals:

Conference Planning Committee

Robert Kirschner, *Conference Coordinator*
Northeastern Illinois Planning Commission, Chicago, IL

Thomas Davenport, *Project Officer*
U.S. Environmental Protection Agency, Region 5, Chicago, IL

Dale Bryson, Camp Dresser & McKee, Naperville, IL
Martin Kelly, Southwest Florida Water Management District, Tampa, FL
Bruce Kirschner, International Joint Commission, Detroit, MI
Lyn Kirschner, Conservation Technology Information Center, W. Lafayette, IN
Eric Livingston, Florida Department of Environmental Protection, Tallahassee, FL
Richard Mollahan, Illinois Environmental Protection Agency, Springfield, IL
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Securing the Urban Greenfrastructure: Integrating Stormwater Management with Regional Growth Management

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Introduction

As a representative of the Coalition for a Livable Future[1], I was asked to present information on what we are doing in the Portland, Oregon and Vancouver, Washington metropolitan region to integrate the "Urban Greenfrastructure" into our growth management strategies. I will first describe the context in which we are working to integrate more progressive stormwater management and Greenspace (natural area) protection into regional growth management strategies. Then I will discuss the efforts of the Coalition for a Livable Future to further integrate that work into a framework that includes social and environmental growth objectives.

The Portland Park Bureau's 1903 master plan contains the following admonition to utilize the natural landscape to address issues of water resource management:

Marked economy may be effected by laying out parks, while land is cheap, so as to embrace streams that carry at times more water than can be taken care of....thus, brooks which would otherwise be put in large underground conduits at enormous public expense, may be made attractive parkways.

This has a certain Olmstedian ring to it, but it was John Charles Olmsted not his father who first articulated a policy of multi-objective stream management some 95 years ago. While there may be no such thing as "cheap" land anymore, especially in the cities, realizing Olmsted's vision is very much within our ability to implement in the urban and urbanizing environment. That is the path we have set out on in the 24 cities and three counties of the Portland metropolitan region.

Building and Retrofitting Livable Regions

One of Henry David Thoreau's most quoted statements is, "In wildness is preservation of the earth." Ironically, some members of the conservation community, carrying Thoreau's aphorism into battle, have contributed to the unfortunate demonization of the city. Some in the conservation community, I believe, have also deified the so-called "American Dream" of owning a quarter acre, or better yet, a rural homesite in which to commune with nature, as if nature cannot be appreciated in an urban setting. Of course, most of them will then commute to the much-derided city to work. The resultant urban sprawl has consumed vast acreages of prime farm land and productive forest land; fragmented wildlife habitat; destroyed a sense of community; created expanding areas of concentrated poverty in inner cities; and significantly increased the cost of infrastructure, including stormwater management.

Robert Liberty, Director of 1000 Friends of Oregon, provided the following data which illustrate the tremendous consumption of land that is the signature effect of unfettered urban sprawl. Between 1970 and 1990 the Chicago region's population grew by 4% but its land area increased by 50%. Kansas City's population grew by 29% during that same period and its land consumption was 110%. Michigan's population is projected to grow by 12% between 1990 and 2020 while the urbanized areas in that state will increase between 63% and 87%. A study commissioned by the New Jersey legislature concluded that low-density development consumed 130,000 more acres than a more compact urban form would have, at an additional cost of $740 million for roads and $440 million for sewer and water infrastructure.

Perhaps Thoreau's adherents would be better served by a new aphorism, "In livable cities is preservation of the wild." It will only be through the creation or, where necessary, the re-creation of livable cities that we will successfully protect the American landscape and the wilderness. But we cannot hope to create compact, land-conserving, urban forms unless we also ensure our cities are places people want to live, not flee. Without a vibrant, healthy urban Greenfrastructure (an interconnected system of streams, wetlands, Greenspaces and greenways), we will not create, or recreate—retrofit, if you will—livable cities.

Smart Growth and Urban Stormwater Management

There is a growing national movement toward compact urban form, although in truth it is not so much a movement

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forward as back to a development pattern that is reminiscent of our pre-World War II, non-auto-dominated communities. The weakness of this new Smart Growth movement is the lack of an explicit nexus between higher-density, mixed-use, pedestrian-friendly development and redevelopment on the one hand, and the protection and long-term management of the urban Greenfrastructure on the other. I was recently discussing the Smart Growth movement with one of its adherents in Washington, D.C. and noticed a huge, four-foot by six-foot poster on the wall. Amidst the multi-modal transit schemes, row houses, townhouses and mixed-use developments was a small, three- by five-inch area marked "open space." There were no wetlands, no un-culverted streams, not even a tree in this Smart Growth scenario.

How do we rectify this? First, we can ensure that the next version of that poster has not only the progressive urban planning icons, but also urban waterways with healthy riparian zones, parks that serve multiple purposes–including stormwater and floodplain management–and waterways used by people, fish and wildlife. To promote this vision we need to form new partnerships between non-government organizations (NGOs) and the practitioners of water resource, stormwater and floodplain management. We also need to build new coalitions among NGOs and the grassroots citizen groups that can promote the integration of urban waterway management into the Smart Growth movement.

The Coalition For a Livable Future has successfully brought together unlikely partners in the nonprofit community to integrate stormwater management into local and regional land use programs, and to integrate environmental issues with social and environmental equity concerns. The Coalition For a Livable Future (CLF) is a group of more than 40 nonprofit organizations, working in the Portland-Vancouver metropolitan region, including: the Urban League of Portland which represents low-income communities and people of color; the Community Development Network, an umbrella organization for the region's affordable-housing advocates; Bicycle Transportation Alliance and other alternative transit advocates; several stream groups and watershed councils; and three local neighborhood associations. What many would consider more "mainstream" conservation organizations such as the Audubon Society of Portland and 1000 Friends of Oregon are also CLF members.

Coalition Building: Linking Environmental and Social Concerns to Regional Growth Management

Robert Liberty, director of the 1000 Friends of Oregon, provided the catalyst for the formation of the coalition by bringing Myron Orfield, a Minnesota state legislator, to Portland. Representative Orfield has studied metropolitan regions throughout the U. S. and has documented the "hollowing out" of their urban cores. His maps graphically illustrate the economic disparity that develops between communities as the rapidly growing, sprawling suburbs capture a larger share of the regional tax base—where demand for social services is lowest—while urban neighborhoods with the highest social needs struggle to meet a high demand for services, with a dwindling tax base.

The containment and the reversal of these phenomena was the primary basis for formation of the CLF. While the Portland metropolitan region does not exhibit all the symptoms of urban decline observed throughout the U. S., there were enough signs that we might be headed down the same path of metropolitan decay. The result of Orfield's presentation and subsequent meetings was the writing of a mission statement and development of core principles around which diverse partners could join to become a regional coalition. The coalition's mission statement and objectives were sent to interested organizations and individuals with an invitation to join. Every member organization has been asked to sign an agreement to work not solely on their individual issues, but to commit to promoting the entire integrated package of CLF objectives.

CLF's mission is: *To protect, restore, and maintain healthy, equitable, and sustainable communities, both human and natural, for the benefit of present and future residents of the greater metropolitan region.* The focus of the coalition is to adapt or change government land use, transportation, housing, public investment, and economic and environmental policy through advocacy, research, and public education.

The CLF's objectives are:

1) Protect the region's social and economic health including: preventing displacement of low and moderate income residents and people of color; assuring equitable access to employment and affordable housing throughout the region; and reversing polarization of income.

2) Develop a sustainable relationship between human residents and the region's ecosystems by: changing patterns of urban expansion to more compact neighborhoods; expanding transportation options; and protecting, restoring and maintaining healthy watersheds, fish and wildlife habitat, and Greenspaces both within and outside the Urban Growth Boundary.

3) Assure fair distribution of tax burdens and government investment within the region.

4) Promote a diverse and tolerant society.

5) Increase public understanding of regional growth management issues; develop effective democratic discourse; and promote broader citizen participation in decision-making regarding regional growth issues.

In forming the CLF, we have brought together affordable-housing advocates, those working in the jobs-with-justice arena, and representatives from low-income com-

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munities and people of color with the land use and transportation specialists. We are not focusing our attention on urban stormwater management alone, but on our regional growth management program, Region 2040[2], cross-interest education in regional growth issues has been such a tremendously powerful political tool that affordable-housing experts testify before local and regional governments supporting Greenspace protection. By the same token, elected officials hear about affordable housing and urban design issues from the Audubon Society of Portland.

Regional Growth Management: The Context for Coalition Building and Stormwater Management

In addition to Orfield's catalytic role in the formation of the CLF, we were fortunate in having a regional planning program to help focus our energy and develop jointly held principles and policies. Metro, the only directly elected regional government in the United States, has authority over the 24 cities and three counties in the Portland metropolitan region. Metro's charter requires it to undertake regional growth management planning and other issues "of regional significance." Water resource management is one of the regionally significant issues that Metro is required to address, as is housing, transportation, hazard mitigation and, with considerable assistance from coalition members, Greenspaces or natural area acquisition and management. The CLF supports Metro's work where it is coincident with our mission and objectives and addresses deficiencies where necessary. One of the initial deficiencies was weak stormwater and watershed management policies.

To date, the CLF has succeeded in persuading Metro's seven-member council to adopt provisions for fair share, inclusionary zoning for affordable housing (which is, as you might suspect, a controversial issue among local governments); low-income community economic revitalization language in the Regional Framework Plan: and newly adopted floodplain and water quality management regulations that will be applied consistently throughout the metropolitan region. Additional acquisition of natural areas, Greenspaces, and implementation of a regional Greenspaces master plan is also a key element of the framework plan.

Greenspaces to Stormwater Management; Securing the Urban Greenfrastructure

One of the first areas of focus for the coalition was participation in the development of a regional vision. Metro's Future Vision Commission developed, among numerous other recommendations, the following vision for the region:

Integrate urban, suburban, and rural lands in a watershed-wide perspective to ensure reduction in downstream flooding, reduction in wintertime flows and enhancement of summer flows, protection of riparian corridors and wetlands and restoration of fisheries. Any future development within the targeted urban reserves must be sensitive to increased stormwater runoff, erosion, and sources of pollution and flooding downstream communities. An integrated, multiobjective floodplain management strategy shall be developed which recognizes the multiple values of stream and river corridors including: enhanced water quality, fish and wildlife habitat, open space, increased property values, education, flood reduction, aesthetics, and recreation. An interconnected system of streams, rivers, and wetlands that are managed on an ecosystem basis and restoration of currently degraded streams and wetlands are important elements of this ecosystem approach.

We next took on the task of redefining what the region viewed as "infrastructure" in our Regional Growth goals and objectives. We developed an alternative definition, took it to the regional advisory committee of local elected officials and the full Metro Council and the following definition of urban infrastructure was adopted:

Infrastructure: Roads, water systems, sewage systems, systems for storm drainage, telecommunications and energy transmission and distribution systems, bridges, transportation facilities, parks, schools and public facilities developed to support the functioning of the developed portions of the environment. Areas of the undeveloped portions of the environment such as floodplains, riparian and wetland zones, groundwater recharge and discharge areas and Greenspaces that provide important functions related to maintaining the region's air and water quality, reduce the need for infrastructure expenses and contribute to the region's quality of life.

From Greenspace Acquisition to Watershed Management

Even prior to the formation of the coalition, the Audubon Society of Portland and several other groups like The Wetlands Conservancy had worked to create a regionally, interconnected natural areas system. The Coalition for a Livable Future identified Metro as the logical government entity to house a regional natural areas system. Working with numerous citizen groups and local park providers, the coalition was able to persuade Metro Council to establish a Regional Parks and Greenspaces Program at Metro.

Again, coalition-building and partnerships with government agencies at every level were key to this successful grassroots effort. We also had to be creative. We brought in "outside experts" such as Dr. David Goode, Director of the London Ecology Unit in England and *New Yorker* author, Tony Hiss, who wrote about our efforts in national publications. We also invited nationally syndicated columnist Neil Pierce to address our newly established coalition of Greenspace advocates, FAUNA (Friends and Advocates of Urban Natural Areas). We then organized two field tours of the East Bay Regional Park District in Alameda and Contra Costa Counties in the San Francisco Bay area so local elected officials and park professionals in our region could see how a regional park system focused on natural areas can be developed and managed.

In spring of 1989, with funding from the Audubon Society of Portland, local neighborhood groups, U. S. Fish and

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Wildlife Service, the U. S. Army Corps of Engineers and a host of other cooperators, Metro commissioned an infrared photography project for the entire Portland-Vancouver metropolitan region, an area covering 1925 square miles (55 miles north to south and 35 miles east to west). Dr. Joe Poracsky, professor of geography at Portland State University, and his graduate students then digitized this low-level imagery to produce for the first time in our region a map of all remaining natural areas in the Portland-Vancouver metropolitan region[3]

The result of these efforts was the development of the political will and broad public support at both the local and regional level to establish a regional Greenspaces program at Metro and to pass, with 60% voter approval, a $135.6 million bond measure (from an increase in property taxes) to acquire up to 6,000 acres of Greenspaces. While this acquisition program is a very important tool, the acquisition of 6,000 acres in a region that contains 204 square miles and measures 38 miles east to west and 26 miles north to south is inadequate to protect the regional landscape.

Regulatory Approach: Region-Wide Floodplain and Water Quality Management

During the past three years the Coalition's Natural Resources Working Group has focused its efforts in the regulatory arena and the development of a region-wide Functional Plan, one element of which–Title 3–addresses floodplain and water quality management. Every opinion survey demonstrates tremendous public support for additional regulatory approaches to the protection of water quality and the region's urban waterways. Water quality is viewed as essential to the maintenance of the region's livability and long-term economic health. Protecting urban streams is consistently rated one of the top values in Metro's public surveys: 60 % of the respondents want to protect urban streams, even if it means limiting development.

The Portland metropolitan region has 213 miles of 303 (d)-listed streams and rivers (water quality limited). In addition to these polluted stream miles, 388 miles of streams have "disappeared" by being culverted, routed underground or piped under streets and parking lots. An estimated 8,840 household units in the region are in, or close to, floodplains. Approximately 1,080 units were built in floodplains since 1992. During the February 1996 flood, 189 homes in the region were inundated with water. According to the Oregon Emergency Management Office, the cost of this flood was about $60 million for the three counties in Metro's jurisdiction.

To address these issues the Coalition For A Livable Future has worked with local stream groups and watershed councils, and with Metro staff and elected officials at the local and regional levels to develop a region-wide strategy to address development in the region's floodplains and the degradation of water quality in the Willamette River and its tributaries. One of Metro's most important advisory committees recently recommended to Metro Council that the region's cities and counties be required to do the following, as one element of the region's integrated Growth Concept:

1. Prohibit new development in the floodplains of the region's rivers and streams or, at a minimum, require "balanced cut and fill."

2. Adopt water quality performance standards that focus on retention of vegetated corridors along all of the region's streams, rivers and wetlands. Each city and county will be required to maintain vegetated corridors which provide shade, stabilize banks, trap soil and other runoff before it enters the water and moderate stormwater flow. The vegetated corridors will measure (on each side of the water feature):

   15' for seasonal streams that drain between 50 and 100 acres, on slopes of less than 25%

   50' for perennial streams or rivers that drain more than 100 acres, wetlands and year-round springs if they are in areas where slopes are less than 25%

   200' for streams and wetlands where slopes are more than 25%

3. Adopt Metro's map which delineates all floodplains, wetlands, stream corridors and steep slopes (over 24%) throughout the region or develop local maps which "substantially comply" with Metro's maps.

4. Adopt region-wide erosion control for any new development (no acreage limitation).

5. Adopt Metro's Model Ordinance or develop a local ordinance which substantially complies with Metro's Model Ordinance.

This new regulatory package will be voted on by the full Metro Council in April of this year (1998). Once adopted (scheduled for May of 1998) local jurisdictions will have up to eighteen months to implement the provisions of the Floodplain and Water Quality Management Functional Plan. The recent listing of steelhead by the National Marine Fisheries Service for the lower Willamette River and the Sandy and Clackamas Rivers, all of which are in Metro's jurisdiction, has brought the Endangered Species Act to the Portland metropolitan region in a manner that will assist in the adoption of water resource-oriented growth management policies.

For example, Oregon's Governor, John Kitzhaber and agency directors from the departments of Agriculture, Land Conservation and Development, Division of State Lands, Water Resources, Fish and Wildlife, Geology and Mineral Industries, and Environmental Quality and Water Resources submitted a joint statement that Title 3 of the Coalition's Functional Plan is *an important first step in watershed enhancement...The recent federal endangered species listing of steelhead in the Columbia and Willamette River systems elevates the significance of habitat protec-*

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tion practices at the local government level. . .We offer the following additional recommendations: add a provision for setback buffers in headwater areas, preferably a minimum of fifty feet. We encourage Metro's early adoption of strong Title 3 policies and implementing measures so that progress can be made soon on the larger work envisioned by chapters 4 and 5 of Metro's Regional Framework Plan. A letter of this nature is unprecedented in the Portland metropolitan region.

Next Steps

The Coalition will continue to focus its efforts on the chapters of the Regional Framework Plan that require consistent, region-wide stormwater management: mandated comprehensive watershed planning for all the region's watersheds within Metro's jurisdiction; development of policies to reduce landslide hazards; and development of a regional fish and wildlife habitat protection program that would ensure an adequate program in every city and county within Metro's jurisdiction. We will also work to implement the Greenspaces Master Plan, which will include an update of the 1989 infrared Greenspaces inventory, and establish plans for a regional interconnected Greenspace system based on maintaining the region's biodiversity and wildlife corridors.

References

1. More information on the Coalition For a Livable Future can be obtained at the Coalition's offices at 534 SW Third Avenue, Suite 300, Portland, OR 97204 (phone: 503-294-2889, email: zack@friends.org).

2. For more information about Metro's Region 2040 growth management planning process contact: Elaine Wilkerson, Director Growth Management Services, Metro, 600 NE Grand, Portland, OR 97232.

3. Metro has a Growth Management Hotline, 503-797-1888 and a website, www.metro-region.org. For more information about their GIS mapping, contact Metro, Data Resource Center 503-797-1742 or Metro's website,

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The Use of Retention Basins to Mitigate Stormwater Impacts to Aquatic Life

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Abstract

Physical habitat and biological measurements were taken in nontidal streams below eight stormwater management pond facilities (BMPs) during the spring of 1996. Two of the sites were predominantly in commercial land use while the remaining six sites were in residential land use. The results were compared to 33 sites with no stormwater controls. Three replicate macroinvertebrate samples were collected in riffle habitats using a kicknet. Biological quality was determined from six metrics using 100-organism subsamples identified to the species level. Physical habitat quality was determined from 12 metrics that defined the condition of the channel, stream bank, and riparian zone. These biological and physical habitat metrics were compared with mean values derived from three reference sites to produce summary index scores for each site, reported as "percent of reference." The overall macroinvertebrate community, as measured using a composite of all six biological metrics (Community Index), was not significantly different between BMP and non-BMP sites. A similar result was found using a composite of three metrics that characterized pollution-sensitive organisms (Sensitive Species Index). The BMPs did not prevent the almost complete loss of sensitive taxa (e.g., mayflies, stoneflies, and caddisflies) after development. Further, the BMPs did not attenuate the impacts of urbanization once the watershed reached 20% impervious cover. Data are needed to determine whether these controls would attenuate impacts at lower levels of development (5-15% impervious cover). Half of the BMP sites had Habitat Index scores comparable to the reference condition, indicating mixed results with regard to the effectiveness of the BMPs in protecting physical habitat. These results suggest that following management actions may be needed: (1) modifications to traditional urban designs that reduce impervious cover and preserve natural features (e.g., "conservation design"), (2) modifications to stormwater retention basin designs (e.g., expanded capacity, constructed wetlands), and (3) the restoration and preservation of forest cover along stream channels, especially along intermittent streams and first and second order perennial streams.

A data set of this size should not be used to derive definitive conclusions regarding the ability of stormwater controls to protect aquatic life and physical habitat. This study characterized the condition of only eight sites, and the stormwater management design criteria varied between the sites. The lack of comparable studies of other regions of the U.S., and the use of other measures of stream condition below stormwater controls suggest the need for additional research.

Introduction

Over the last 90 years, the population of the United States has increased 300%, from 76 million in 1900, to 249 million in 1990 (United States Census 1996). This period has also seen a dramatic shift in the way people live and use the land. In 1900, the majority of the U.S. population (60%) lived in rural areas, while in 1990 the majority (75%) lived in urban areas. This trend continues today although at a slower rate. But even as the rate levels off, roughly three-fourths of the estimated 25 million people that will be added to the population over the next decade will likely live in urban areas. Delaware's population has experienced a similar rate of population increase (185,000 to 666,000) and shift in land use over this period.

This change in demographics and land use has brought about profound changes in the physical, chemical, and biological integrity of nontidal streams in Delaware. The objective of this research was to determine the effectiveness of stormwater controls, principally retention basins, to protect stream resources after urbanization. This study focused on wadeable, nontidal streams and the use of macroinvertebrates and physical habitat as indicators of stream ecological health.

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Impervious surfaces (roads, parking lots, rooftops, driveways, sidewalks, etc.) increase peak flows during storm events and reduce base flows during droughts. Urbanized watersheds with 20-30% impervious cover were found to have 10-15 times the frequency of small flood events (1-year recurrence interval) compared to nonurban watersheds; large flood events (100-year) doubled in size after urbanization (Hollis 1975). This change in stream hydrology affects the physical structure and stability of stream channels through accelerated erosion and sediment deposition. The replacement of native riparian vegetation (e.g., trees) with lawns, parks, golf courses, and structures (e.g., buildings, bridges) along stream channels and floodplains further impact the geomorphology of urban streams.

Water quality contaminants in stormwater (metals, nutrients, organics) further stress aquatic life. Exceedences of dissolved oxygen (DO) criteria occur in streams and ponds through nutrient enrichment and the removal of shade. In a recent survey, unshaded stream channels in Delaware exceeded the State's acute criteria for DO and temperature 73% and 38% of the days, respectively, during the Summer of 1993 (Maxted et al., 1995). Both physical and chemical factors associated with urbanization contribute to the overall biological condition of urban streams.

Aquatic organisms, principally fish and macroinvertebrates, are commonly used to assess the ecological condition of streams, and several researchers have used them to assess the impacts of urbanization (Shaver and Maxted 1995, Jones and Clark 1987, Klein 1979, Limburg and Schmidt 1990, Pedersen and Perkins 1986, Booth and Jackson 1994, Weaver and Garmen 1994, and Garie and McIntosh 1986). These studies have recently been summarized (Schueler 1994). What is generally lacking are studies which use aquatic organisms to evaluate the effectiveness of stormwater controls.

The water quality impacts of stormwater runoff are fairly well documented, as is the ability of a variety of stormwater management facilities to provide water quantity control and water quality treatment. There has been an inherent assumption that water quality treatment and pollutant capture directly translate into aquatic life protection. This assumption has never been validated. In addition, stormwater treatment facilities effectively remove pollutants, but the level of performance is highly variable and needs to be expressed in ranges rather than in specific levels of treatment. At great expense, large amounts of data covering many stormwater events and constituents are needed to make reasonable statements regarding the performance of BMPs in removing pollutants (Urbonas, 1995).

Water quality data can also present problems in terms of data accuracy. Stormwater management facilities often have multiple inflow points and may receive overland flow which makes data collection difficult. Monitoring each inflow point and the facility outfall increases the potential for error in data collection and analysis. Coupled with the need to sample multiple storm events over different seasons and different years, these factors make it difficult to accurately assess the performance of the BMP. These factors also affect the overall cost of monitoring.

What is needed is a simple, long-term approach to system assessment which minimizes the cost of data collection and provides a framework for evaluating the effectiveness of controls. Presented in this paper is a framework for assessing the ecological health of aquatic ecosystems and the performance of stormwater facilities using living resources, in this case aquatic macroinvertebrates. While evaluating stream ecological health using aquatic organisms is widespread, the evaluation of stormwater facility performance using this approach is fairly new. Preliminary results of the present study have been summarized previously (Maxted and Shaver, 1997).

Methods

The heavily urbanized piedmont region of northern Delaware was selected for study. Data collected at 33 sites with no BMPs in the catchment were compared with eight sites sampled below modern stormwater retention basins (Figure 1). As of 1984, about half the piedment region (48%) was in urban land use, 33% was undeveloped, and 19% was in agriculture. Stormwater controls have only recently been included as part of new developments in the region. Therefore, the data collected at the 33 non-BMP sites represent conditions that existed before the implementation of regulatory programs for controlling stormwater runoff. The land use conditions in the watersheds above the 33 non-BMP sites covered the full range of urban land use from relatively undeveloped watersheds with less than 10% impervious cover to heavily urbanized watersheds with greater than 30% impervious cover. Sampling sites were

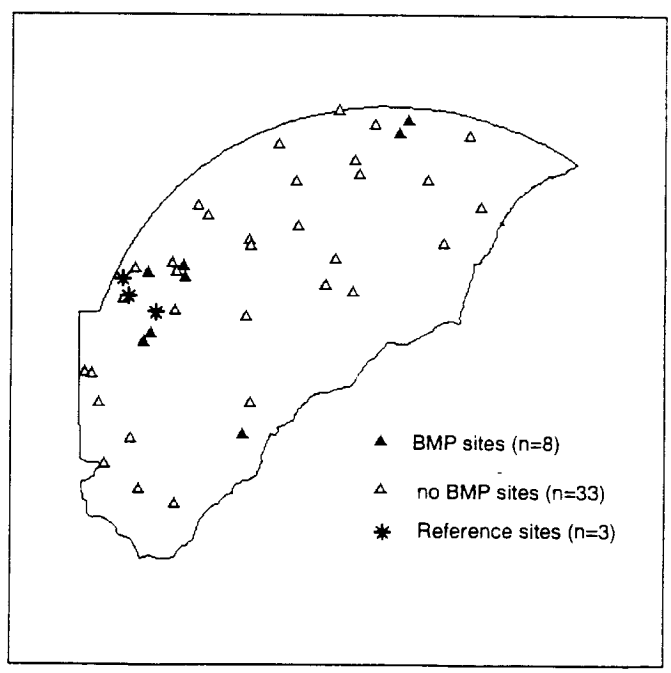

Figure 1. Locations of sampling sites within the northern piedmont region of Delaware.

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located 100 meters below the BMP discharge to minimize the immediate influence of the discharge on the stream and the influence of construction and maintenance activities related to the BMP itself (grading, mowing, habitat disturbance, etc.).

The 33 non-BMP sites were sampled in the fall of 1993, while the eight BMP sites were sampled in the spring of 1996 (between May 2nd and June 6th, 1996). The metrics used to summarize the biological data were not considered to be sensitive to seasonal differences between the fall and the spring, and thus allowed for this comparison.

Macroinvertebrate samples were collected using a 1-meter^2 kick net (750 μm mesh). Each sample was a composite of two collections of a 1-meter^2 area of riffle, combined in a sieve bucket (600 μm mesh). Three replicate collections were made at each site while moving progressively upstream. A single 100-organism subsample was removed from each sample and identified to the species level. Six metrics were derived for each sample: taxonomic richness (TR); richness of the orders of ephemeroptera, plecoptera, and trichoptera (EPT); % EPT abundance (% EPT); % Chironomidae (% C); % dominant taxon (% DT); and the Hilsenhoff Biotic Index (HBI) (Table 1).

Habitat quality assessment included measures of the channel, stream bank, and riparian zone. Each assessment consisted of the visual characterization of a 100-meter segment of the stream using the following 12 parameters. Numerical scores, out of a possible 20 points, were assigned to each parameter.

CM - channel modification: the degree of engineering of the channel shape (e.g., channelized) and the extent to which it meanders.

BSC - bottom substrate/available cover: the amount and variety of submerged stable habitat throughout the stream segment (e.g., riffles, logs, snags, aquatic plants, root-wads along banks, etc.).

E - embeddedness: the degree to which the substrate is surrounded or covered by fine sediment.

Table 1. Biological Metrics Used to Derive Summary Index Scores for BMP and non-BMP Sites.

Metric Name	Description	Type
taxonomic richness	total number of unique taxa	richness
EPT* richness	total number of EPT taxa	rich/tolerance
% EPT abundance	% of sample that are EPTs	tolerance/comp
% dominant taxon	largest % of a single taxon	composition
% Chironomidae**	% of sample from this group	tolerance
Hilsenhoff (HBI)	composite tolerance by taxon	tolerance

* EPT - the orders ephemeroptera (mayflies), plecoptera (stoneflies), and trichoptera (caddisflies); high richness or relative abundance indicates high quality.

**Chironomidae - family of midges; high relative abundance indicates low quality.

RQ - riffle quality: the dominant substrate found in riffles; cobbles are the most desirable, boulders and gravels are the least desireable.

FR - frequency of riffles: the abundance of riffle areas in the stream segment.

SD - sediment deposition: the degree to which new sediment is deposited in the stream channel as evidenced by islands, point bars, and sand and silt covering stable habitats.

V/D - velocity/depth: the presence of four categories of flow regime; slow and deep, slow and shallow, fast and deep, fast and shallow.

BS - bank stability: the proportion (%) of stream banks that show evidence of recent and active erosion.

BV - bank vegetative type: the dominant vegetation on the stream bank; trees and shrubs being most desirable, grasses being the least desirable; left and right banks scored separately and then combined.

S - shading: the percent of the stream surface that is shaded throughout the day.

RZ - riparian zone width: the width of the riparian zone showing little or no evidence of human activity; left and right sides scored separately and then combined.

HCI - Habitat Comparison Index: summary index of habitat quality; individual parameter scores summed and divided by a reference value; index values expressed as "percent of reference."

Three summary index scores, two biological and one habitat, were derived for each site following procedures developed by EPA (Plafkin, et al., 1989). Three reference sites were sampled during the same seasonal period as the sampling sites and used to derive index scores reported as "percent of reference." Habitat Index scores were determined by comparing the total habitat score for each BMP site with the mean total score for the three reference sites. Community Index (CI) scores were determined by comparing all six biological metric values for each site with the mean values from the three reference sites. The CI was used to define the overall quality of the macroinvertebrate community. The Sensitive Species Index (SSI) scores were determined using the three biological metrics (EPT, % EPT, and HBI) that define the components of the community that are the most sensitive to organic pollution. Mean CI and SSI scores for each site were determined from the CI and SSI scores from the three replicate samples.

The biological data were plotted against % impervious cover estimates determined for the catchment above each site. Land use was determined from digitized 1992 land use data. Percent impervious cover estimates were made by multiplying the area of each land use category by the % impervious cover estimate for that category, as published by the U.S. Department of Agriculture (USDA 1986), sum-

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ming the values for all the land use categories, and then dividing the total % impervious area by the total area for the catchment. The CI biological index values were plotted against the HCI habitat index values to further characterize the habitat quality of the BMP sites. Mean values differing by more than one standard deviation were defined as statistically significant.

BMP Site Selection

BMP site selection employed a variety of information sources including an existing stormwater facility inventory and discussions with individuals familiar with a number of facilities. The stormwater facility and stream criteria used to select BMP sites are listed below. Eight sites met these criteria and were selected for study; two sites were predominantly in commercial land use while the remaining six sites were in residential land use (Table 2). The two commercial sites and one residential site met modern design standards for peak control of the two-, ten-, and 100-year storms, and extended detention of the first inch of runoff. Four residential sites were designed for control of the ten-year storm only. One residential site was located in the main channel of Jenny's Run below five separate retention basins that captured approximately 75% of the the urban land use in the catchment (Table 2).

Stormwater facility criteria:

- Facilities had to be retention or detention basins, so that one specific type of BMP (e.g., ponds designed for stormwater control) could be evaluated.
- To the greatest extent possible, the facility had to meet current design criteria which included peak rate control (two-, ten-, 100-year storms) and water quality performance (24-hour detention for the first inch of runoff). If a sufficient number of facilities were not found, due to the recent nature of the State Stormwater Management Program (effective date July, 1991), older retention ponds serving a development were considered.
- Facilities had to be at least two years old. The concern with newer facilities was that there was potential for construction-related stream impacts. If a new facility had significant instream impacts, the cause of the impact might be related to excess runoff and sedimentation during construction rather than the performance of the BMP.
- Impervious cover in the catchment to the facility had to be at least 20%. This would answer the initial question concerning the effectiveness of BMPs in already urbanized areas. Based on the results of this study, future studies might address the question of the effectiveness of BMPs at earlier stages of urbanization (e.g., 5-15% impervious cover).

Receiving stream criteria:

- Discharge from the BMP represented the predominant flow in the stream.
- Riparian zone had native vegetation (e.g., trees, wooded, and shaded) and was not directly impacted by human activities. This criterion might be difficult to achieve in the heavily urbanized piedmont region of Delaware.
- The receiving stream had perennial flow. This criterion might be difficult to achieve since the streams below individual retention basins are first order streams with fairly small drainage areas.
- Riffles with a cobble substrate were common.

Results

The mean EPT richness (EPT), % EPT abundance (% EPT), % Chironomidae (% C), and Hilsenhoff Biotic Index (HBI) metrics were significantly different between the BMP sites and the reference sites (Table 3). This indicated that none of the BMP sites prevented a shift in the macroinvertebrate community from one dominated by pollution-sensitive organisms to one dominated by pollution-tolerant organisms. Taken together, sites below BMPs had a low proportion of the pollution-sensitive organisms (14% EPT) and a high proportion of pollution-tolerant organisms (54% Chironomidae), while the community at reference sites was almost exactly the opposite (Table 3). The BMP sites had half the mean HBI value of the reference sites, indicating a shift at the species level as well. Similar results were found using the two summary biological indi-

Table 2. BMP Facility Data

Site	Land Use	% Impervious Cover	Drainage Area	Development
BMP 1*	Residential	25	88.0 acres	Corner Katch
BMP 2**	Commercial	22	83.0 acres	Brandywine Com
BMP 3**	Commercial	65	36.0 acres	Core States
BMP 4**	Residential	30	32.0 acres	Hunt at Louviers
BMP 5*	Residential	28	383.0 acres	Veranda
BMP 6*	Residential	31	107.0 acres	Limestone Hills
BMP 7*	Residential	30	157.0 acres	Chestnut Hills
BMP 8***	Residential	23	330.0 acres	Jenny's Run

* Project design based on peak control of the ten-year storm only.
** Project design based on peak control of the two-, ten-, and 100-year strom, in addition to 24-hour extended detention for the first 1" of runoff.
*** Site in main stream of Jenny's Run; considered stormwater flow from more than one development site.

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Table 3. Mean Values for Six Biological Metrics and Two Summary Indices Below Eight BMPs; Taxonomic Richness (TR), EPT Richness (EPT), % EPT Abundance (% EPT), % Chironomidae (% C), % Dominant Taxon (% DT), Hilsenhoff Biotic Index (HBI), Community Index (CI), and Sensitive Species Index (SSI); Standard Deviation Appears in Parenthesis.

Site	samples N	TR	EPT	% EPT	% C	% DT	HBI	CI	SSI
		Biological Metrics						(% of reference) Summary Indices	
BMP 1	3	35 (5)	7.3 (0.6)	18 (6)	54 (6)	15 (5)	5.2 (0.4)	49 (3)	26 (6)
BMP 2	3	21 (6)	2.0 (1.7)	26 (19)	52 (20)	29 (7)	6.1 (0.4)	35 (15)	15 (17)
BMP 3	3	31 (3)	1.7 (0.6)	2 (1)	71 (10)	27 (11)	5.8 (0.4)	33 (3)	7 (6)
BMP 4	3	26 (6)	6.0 (1.0)	17 (9)	60 (22)	27 (14)	5.3 (0.7)	39 (18)	18 (18)
BMP 5	3	19 (3)	0.0 (0.0)	0 (0)	37 (24)	51 (26)	7.1 (0.5)	25 (6)	0 (0)
BMP 6	3	22 (7)	6.7 (1.1)	14 (2)	38 (36)	47 (22)	6.4 (0.8)	31 (9)	7 (13)
BMP 7	1	29 (-)	5.0 (-)	9 (-)	75 (-)	22 (-)	5.4 (-)	35 (-)	11 (-)
BMP 8	3	23 (3)	7.7 (1.5)	26 (8)	60 (8)	20 (5)	4.7 (0.5)	51 (12)	33 (20)
all BMPs	28	25 (7)	4.5 (3.0)	14 (12)	54 (21)	30 (17)	5.8 (0.9)	38 (12)	15 (15)
Reference	10	24 (4)	10.3 (1.8)	56 (12)	14 (13)	24 (4)	2.9 (0.6)	100	100

ces. The mean Community Index (CI) scores for the BMP sites ranged from 25-51% of the reference condition while the mean Sensitive Species Index (SSI) scores ranged from 0-33% (Table 3).

Mean values for the biological metrics and summary indices for the eight BMP sites were compared to 21 non-BMP sites with similar land use (Table 4). Only non-BMP sites with greater than 20% impervious cover were considered in order to provide a similar level of urban development between the two groups of sites. There was no significant difference between the two groups of sites for the EPT, %EPT, and %C metrics, as well as the CI and SSI index values (Table 4). Both the BMP and non-BMP sites were significantly different from the reference condition for most biological metrics, indicating that neither group of sites approximated conditions found in undeveloped watersheds (Table 4). While the BMPs appeared to increase the relative abundance of EPTs (i.e., % EPT), it had no effect on either taxonomic richness of EPTs (i.e, number of unique EPT taxa) or the HBI; both are good indicators of pollution tolerance.

A lack of biological improvement with the eight BMP sites was observed when the data were plotted against % impervious cover. No improvement in biological condition was observed using either the Community Index (Figure 2) or the Sensitive Species Index (Figure 3), as compared to sites without BMPs. BMPs did not prevent the loss of sensitive species found at reference sites. The degree of urbanization did not appear to affect biological conditions at the BMP sites. The one BMP site with 65% impervious cover had a similar biological condition to the seven sites with 22-32% impervious cover.

Half of the BMP sites (BMP2, BMP3, BMP6, and BMP7) had habitat scores less than 90% of reference, indicating physical habitat impairment (Table 5). These sites exhibited the physical characteristics of urban streams with no controls, indicating that the BMPs were not effective at eliminating the impacts of urbanization. The impacts were most often associated with bank instability and channel sedimentation. The other half of the sites (BMP1, BMP4, BMP5, and BMP8) had habitat conditions similar to the reference sites (i.e., greater than 90% of reference). It appeared that some of the BMPs provided stable channel characteristics, although there was no pattern related to BMP design type or drainage area. The three sites that had the highest physical habitat quality (BMP1, BMP4, and BMP8) represented the full range of both BMP design type and drainage area. The level of impairment, both physically and biologically, was also illustrated when biological quality was plotted against habitat quality (Figure 4).

Discussion

Retention and detention basins designed to control stormwater did not protect aquatic life from the adverse

Table 4. Comparison of Mean Values for Six Biological Metrics and Two Summary Indices Between Reference Sites and Sites With and Without Stormwater BMPs; Taxonomic Richness (TR), EPT Richness (EPT), % EPT Abundance (% EPT), % Chironomidae (% C), % Dominant Taxon (% DT), Hilsenhoff Biotic Index (HBI), Community Index (CI), and Sensitive Species Index (SSI); Standard Deviation Appears in Parenthesis.

Site	samples N	TR	EPT	% EPT	% C	% DT	HBI	CI	SSI
		Biological Metrics						(% of reference) Summary Indices	
Reference	10/3	24 (4)	10.3 (1.8)	56 (12)	14 (13)	24 (4)	2.9 (0.6)	100	100
BMP	28/8	15 (7)	4.5 (3.0)	14 (12)	54 (21)	30 (17)	5.8 (0.9)	38 (12)	15 (15)
no BMP*	29/21	20 (5)	4.7 (2.7)	27 (18)	28 (23)	27 (18)	5.1 (1.2)	36 (14)	14 (15)

* only urban sites with 20-65% impervious cover included

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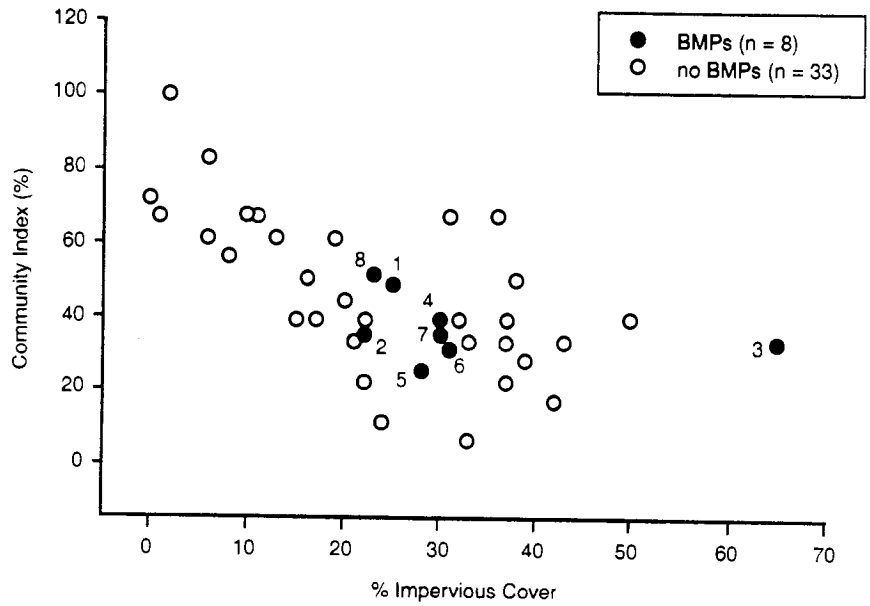

Figure 2. The effects of urbanization on the macroinvertebrate community; numbers denote BMP sites.

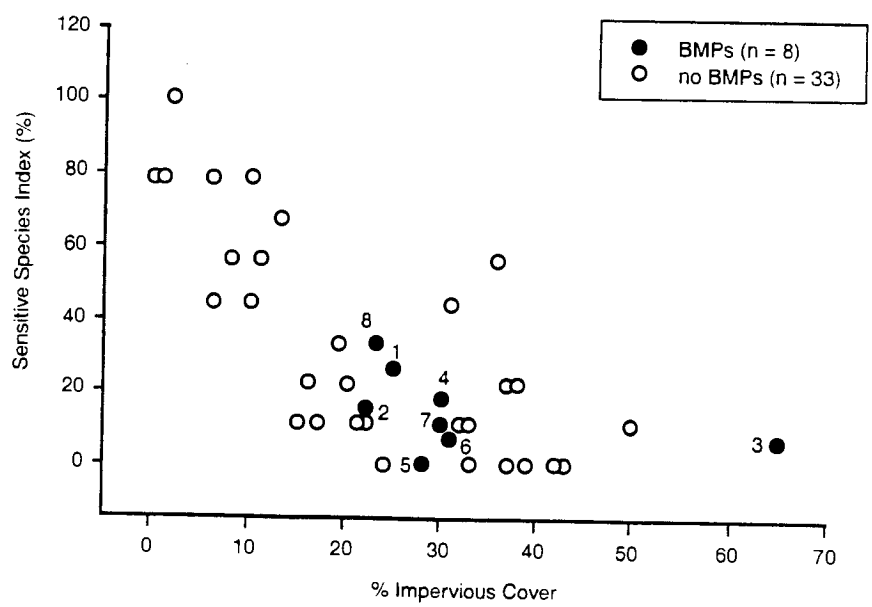

Figure 3. The effects of urbanization on sensitive species of macroinvertebrates; numbers denote BMP sites.

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Table 5. Habitat Metric Scores Below Eight BMPs; Habitat Comparison Index (HCI) Reported as % of Reference; See Text for Abbreviations.

Site #	CM	BSC	E	RQ	FR	SD	VD	BS	BV	S	RZ	Total	(%) HCI
BMP 1	12	19	17	16	18	16	10	13	10	20	20	171	100
BMP 2	20	6	16	13	9	5	7	6	12	18	20	132	77
BMP 3	15	13	11	13	12	7	9	11	10	15	12	128	75
BMP 4	19	18	18	14	19	16	10	18	18	16	20	186	109
BMP 5	19	19	9	11	19	6	10	20	10	16	17	156	91
BMP 6	17	18	12	18	14	8	10	6	4	19	20	146	85
BMP 7	11	5	19	10	1	6	11	20	10	17	13	123	72
BMP 8	18	19	19	17	19	16	10	11	10	20	20	179	105
Reference	17	17	18	15	17	12	10	11	17	16	20	171	100
Possible	20	20	20	20	20	20	20	20	20	20	20	220	-

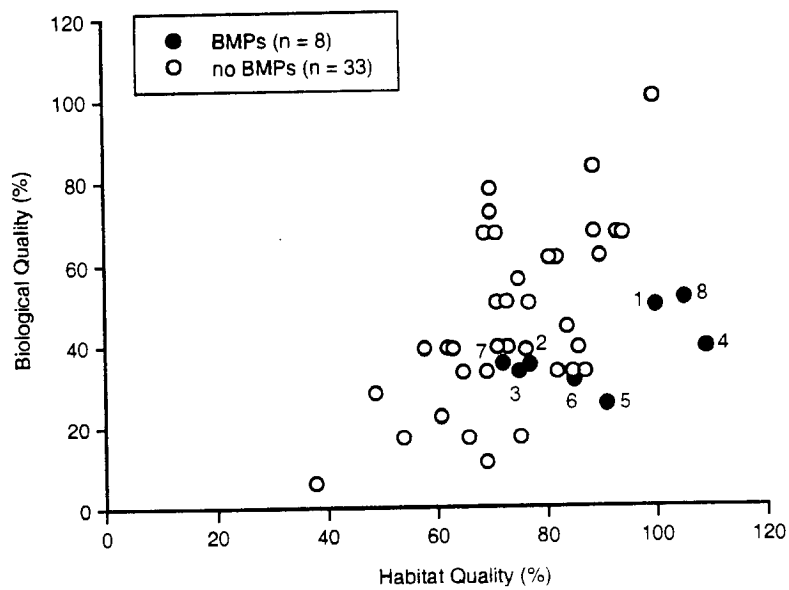

Figure 4. The effect of habitat quality on biological integrity; biological quality measured using the Community Index; numbers denote BMP sites.

effects of urbanization. This may be due to factors related to the design and construction of the BMPs themselves, such as temperature, water quality (road salt, contaminants), inadequate controls during construction, and age of the facility. Modern extended detention basins are not designed to control dissolved contaminants and temperature extremes. Constructed wetlands added on to retention ponds may be needed to remove dissolved contaminants and attenuate temperature extremes.

Half of the BMP sites did not provide habitat conditions comparable to reference areas, indicating that the ponds did not maintain stable channel characteristics. Further study is needed to define the controlling factors. One of the most important factors contributing to both the physical habitat and biological impacts we observed may be related to hydrologic modification of the watershed. Ponds mitigate, to some degree, the hydrologic changes resulting from conventional urbanization, but they cannot reduce the significant increase in total volume of post-development stormwater traveling through the watershed. Efforts to simulate pre-development hydrology (through conservation design) may be necessary to protect aquatic resources.

The impacts we measured 100-200 meters below the facility may be attenuated with distance downstream. There was some indication, from visual observations in the field, that biological conditions improved with increased distance from the discharge. More data are needed to determine the areal extent of the impacts in streams we have reported below stormwater detention facilities.

The effect of the ponds themselves on stream ecology should not be overlooked. The aquatic community is often different upstream and downstream of ponds, even in unde-

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veloped forested watersheds. While one of the three reference sites had a series of old farm ponds in the watershed, two did not. Ponds effectively convert inorganic carbon and nutrients to organic matter through photosynthesis. This fine particulate organic matter (FPOM) discharged to the stream during runoff events is a preferred food source for invertebrates and fish tolerant to organic pollution. The effect on living resources we observed and quantified below the eight BMP ponds may also be related to the construction of the ponds themselves. The important conclusion remains, however, that ponds constructed to treat stormwater were inadequate to protect aquatic life.

Site selection was more difficult than we first anticipated. Many of the streams that received a discharge from a retention or detention pond were too small to sample, had a degraded riparian zone (unshaded), or discharged directly to a larger tributary. Many of the BMP sites not selected for study had small drainage areas that would have had intermittent flow even under pre-development conditions. It was difficult to find a representative sample of stormwater management BMPs that met modern design criteria due to the relative short time between the initiation of the Stormwater Management Program in Delaware and this monitoring effort. The only sites meeting the state's stormwater management requirements in terms of when they were constructed and their design criteria were the two commercial sites and one residential site. The other residential sites were selected because retention ponds are a preferred practice under the new state program.

Construction-related impacts must be expected as a result of increased stormwater discharges and elevated sediment loadings. There has to be a period of time after construction, and before measurements should be taken, to assess the response of a receiving system to site development and stormwater management facilities. We can make recommendations, such as a period of two years used here, but that recommendation must be considered "preliminary" and subject to variation around the country due to differences in climatic and other factors.

The importance of riparian zone protection and restoration cannot be stressed too much. More effective management of riparian zones must be provided if receiving systems are to acheive the structure and function of undisturbed systems. The greatest difficulty in site selection was finding BMPs that discharged to streams with undisturbed riparian areas. This was not surprising given the results of a recent statewide survey in which 87% of the nontidal stream miles were found to have degraded physical habitat (Delaware DNREC 1994). The environmental benefits of control efforts (both structural and non-structural) will be reduced if we fail to restore and protect riparian habitat.

This study represented a different approach to assessing BMP effectiveness as compared to chemical monitoring. Traditional approaches that focus on chemical contaminants determine aquatic life use support based on pounds of pollutants removed and compliance with chemical criteria. Our approach looked directly at the aquatic organisms the controls were designed to protect. Living resources are the only direct measure of aquatic life condition. All others, including physical habitat, are surrogates that may underestimate or overestimate the true condition of living resources.

Some have argued that undeveloped forested watersheds should not be used as the reference condition for evaluating the performance of stormwater controls. They also assert that fundemental changes in land use in urban watersheds justifies the establishment of a lower quality "urban stream standard" for aquatic resources. They feel that this lower standard is needed because streams in urban watersheds will never achieve such a high level of quality even with extensive land use and stormwater controls. Further, they claim that urban land uses impact only a small percentage of stream resources in most regions.

We reject these arguments for several reasons. First, such an "urban stream standard" would be nearly impossible to set. How would such a level of "acceptable" or "achievable" quality be determined? Whatever approach was selected would undoubtedly be influenced by political rather than scientific factors. Second, conservation design practices coupled with structural and non-structural controls do not yet exist over extensive areas. We, therefore, have no way of knowing whether the application of these controls might achieve a higher level of ecological quality. Third, if we were to set a lower standard, we would eliminate a principal incentive for challenging and testing the standard. And lastly, urban areas affect an ever-increasing proportion of nontidal streams, particularly intermittent, first, and second order streams. Urban sprawl affects aquatic resources far away from city centers, extending the proportion of streams affected by urbanization. While roughly half of the piedmont region of Delaware is in urban land uses, it adversely affects nearly all of the 270 miles of nontidal streams.

It is too early to panic, as additional studies are needed. Our results should best be described as preliminary. If further studies confirm these results, BMP design criteria will have to be reconsidered to provide a greater level of protection to receiving systems. Similar data are needed at more sites in the piedmont region of the Mid-Atlantic U.S., before making definitive conclusions on the effectiveness of stormwater basins in protecting aquatic life and physical habitat. Data are also needed for various types of urban designs (e.g., conservation design), various levels of impervious cover, and various types of BMP designs, including the three presented here and the use of constructed wetlands. The ultimate question remains to be answered: Can urban developments that incorporate available control technologies be cost-effective and marketable, while protecting living resources?

Urban Retrofit Opportunities

What are the implications of this research with regard to areas already undergoing various degree of urbanization?

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First, this research indicates that retention basins are not sufficient to protect living resources. It is likely that changes are also needed in the way urban areas are designed and constructed in the first place. It should be no surprise that ponds added on to conventional urban developments, where nearly 100% of the development site is modified for human uses, did not protect aquatic life. The concept of "conservation design" used in conjunction with structural and non-structural controls may be necessary. Conservation design encompasses a range of alternative design practices that reduce impervious surfaces (e.g., reduced roadway width, reduced setbacks), preserve sensitive natural features (woodlots, wetlands, floodplains), and reduce collection system infrastructure (grassed swales, smaller and more numerous retention areas). The State of Delaware, in conjunction with the Brandywine Conservancy, has recently completed a manual on conservation design (Delaware DNREC 1997).

Second, protecting intermittent, first, and second order perennial streams needs to be mentioned since their importance is often overlooked. From a developmental perspective, these streams are often filled, piped, rerouted, or otherwise altered. We then, through regulatory programs, attempt to protect third and higher order streams and their associated resource values. It may not be possible to protect the values of these higher order streams and rivers unless we first protect the headwater streams at the top of the watershed.

Third, the design of stormwater retention basins may need to be modified to enhance their performance, particularly with regard to dissolved contaminants and temperature effects. It should also be no surprise that ponds may not be sufficient, by themselves, to attentuate the water quality, hydologic, and biological effects of urbanization. In fact, the ponds themselves may be contributing to the problem. Conservation design in conjuction with extended detention and constructed wetlands may be necessary to protect stream ecological health. Further, we may want to make a requirement that all ponds be "off-line" from wetlands, intermittent streams, and first and second order perennial streams to prevent the direct impact that the BMPs have on existing aquatic resources.

Fourth, riparian restoration should be implemented immediately, which would provide significant benefits to stream health even in heavily urbanized areas. Most of the BMP sites visited during site selection could not be studied because the streams they discharged to lacked native riparian vegetation (e.g., trees). They most often consisted of backyards or parks with grassed areas and few trees. Through public education and the establishment of easements, these areas should be preserved and restored as natural wooded riparian corridors. This will be necessary, eventually, even after the implementation of structural and non-structural controls, to protect aquatic resources. Since natural revegetation with trees can take many years, efforts should be initiated now.

All of these objectives will need to be met if streams in urban areas are to attain the structure and function of natural systems. Since prevention is often more effective and less expensive than treatment, the most critical watersheds are those in the early stages of urbanization (5-15% impervious cover). The need to focus attention on these watersheds is important because impacts are often permanent once the urban land use is in place. Additional research and monitoring is especially important in these areas while we also attempt to retrofit conditions in already developed watersheds.

Acknowledgments

The authors would like to thank Frank Piorko for providing assistance in selecting sites. Ellen Dickey provided assistance in sample collection, data management, and data analysis. Greg Mitchell and Terry Cole provided assistance in sample collection. The Water Resources Agency of New Castle County produced the estimates of land use.

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Assessing the Status of Aquatic Life Designated Uses in Urban and Suburban Watersheds

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Introduction

The health and well-being of the aquatic biota in surface waters is an important barometer of how effectively we are achieving the goals of the Clean Water Act, namely the maintenance and restoration of biological integrity and the basic intent of water quality standards. States designate water bodies for beneficial uses (termed designated uses) that along with specific chemical, physical, and biological criteria, assure the protection and restoration of aquatic life, recreational, and water supply functions and attributes. Ohio Environmental Protection Agency (EPA) employs biological, chemical, and physical monitoring and assessment techniques to assess the status of these beneficial uses and to satisfy three major objectives:

1) determine the extent to which use designations assigned in the Ohio Water Quality Standards (WQS) are either attained or not attained;

2) determine if use designations assigned to a given water body are appropriate and attainable; and,

3) determine if any changes in key ambient biological, chemical, or physical indicators have taken place over time.

An integrated biological, chemical, and physical monitoring and assessment approach has been used to support all relevant water quality management activities, including urban stormwater issues, within Ohio EPA during the past 18 years. The details of this process have been extensively described elsewhere (Ohio EPA 1987a,b; Ohio EPA 1989a,b; Yoder and Rankin 1995, 1998).

Urban Watersheds

Urban watersheds in Ohio exhibit a familiar legacy of aquatic resource degradation. Few, if any, ecologically healthy watersheds exist in the older, most extensively urbanized areas of Ohio (Yoder 1995) and no headwater streams (i.e., draining <20 mi.^{2}) sampled by Ohio EPA during the past 18 years in these areas have exhibited full attainment of the Warmwater Habitat (WWH) use designation (Yoder and Rankin 1997).

The activities that have the greatest impacts on aquatic life in Ohio's urban watersheds include the wholesale alteration of watershed hydrology, loss and degradation of riparian habitat, direct instream habitat degradation via channelization, culverting, and interceptor sewer line placement, excessive sedimentation resulting from land disturbance activities and stream bank erosion (strongly linked to riparian encroachment), and contributions of excessive nutrients, oxygen-demanding wastes, and toxic chemical pollutants via urban runoff, point source discharges (both permitted and unpermitted), and spills and other releases. According to the 1996 Ohio Water Resource Inventory (305[b] report), urban and suburban sources are responsible for aquatic life use impairment in nearly 1000 miles of Ohio streams and rivers and more than 23,000 acres of lakes, ponds, and reservoirs (Ohio EPA 1997). These activities also threaten existing full use attainment in nearly 160 miles of streams and rivers and may pose a potential problem in more than 4380 miles of streams and rivers that have not yet been fully monitored and evaluated. These are also one of the fastest growing threats as urban and suburban development extends further into rural watersheds.

While much attention has been paid to toxic substances in urban runoff, evidence suggests that sedimentation is the most pervasive *single* cause of impairment associated with nonpoint sources in Ohio. While sediment deposition in lotic and lentic environments is a natural process, it becomes a problem when the capability of the ecosystem to

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"assimilate" the sediment load is exceeded. The effects of sediment on aquatic life are the most severe in the ecoregions of Ohio where: (1) upland erosion and runoff are moderate to high, (2) clayey silts that attach to and fill the interstices between coarse substrates predominate, and (3) streams and rivers lack the ability to expel the finer grained sediments from the low-flow channel because of instream and riparian habitat degradation. Estimates of gross erosion *alone* are not consistently correlated with adverse impacts to aquatic communities, although this is a frequently used indicator for prioritizing nonpoint source management efforts (Yoder 1995).

Bioassessment of Urban Watersheds

Ohio EPA uses biological criteria via a bioassessment approach in the designation and assessment of rivers and streams. Biological criteria are the principal tool for determining impairment of designated aquatic life uses and bioassessments play a central role in the Ohio Nonpoint Source Assessment (Ohio EPA 1990; 1991), the biennial Ohio Water Resource Inventory (305b report; Ohio EPA 1997), and watershed-specific assessments of which Ohio EPA completes from 6-12 each year. Biological criteria represent a measurable goal against which the effectiveness of pollution control and other water quality management efforts can be judged. However, biological assessments must be accompanied by appropriate chemical/physical measures, land use characterization, and source information necessary to establish linkages between stressors and the biological responses.

Methods And Analyses

For bioassessments to achieve their maximum effective use in the assessment of urban streams, a watershed design to sampling and analysis should be employed. A recent example is the Cuyahoga River basin in northeastern Ohio and small, wadeable streams of the Columbus metropolitan area (Franklin County) in central Ohio. The former represents historically and extensively urbanized streams including a mix of residential, commercial, and industrial land use, streams draining recent and rapid suburban development, and larger streams which are dominated by point source effluents, principally treated municipal sewage. The latter case includes small watersheds affected mostly by residential urban land use with a wide range of intensity from older areas to recent and rapidly developed suburban areas.

Biological and Water Quality Assessments

Fish and macroinvertebrates were sampled respectively, at 82 and 48 locations, in the Cuyahoga River basin in 1996, and an additional 32 locations were sampled for macroinvertebrates in 1991. Water samples were collected up to six times at 40 macroinvertebrate sampling locations and 63 fish sampling locations, and included standard field parameters (D.O., temperature, pH, conductivity), nutrient series (N and P), demand parameters (suspended solids, BOD, COD), and selected heavy metals. Drainage areas at Cuyahoga River basin stream sites ranged from approximately 2 to 700 mi^2. Fish communities only were sampled in the Columbus area, at 80 stream locations with drainage areas at all sites less than 35 mi^2. No water chemistry samples were collected. Macroinvertebrate community performance was evaluated using the Invertebrate Community Index (ICI; DeShon, 1995). The ICI is a multimetric index comprising ten attributes of community structure and composition. The individual metrics were scored against expectations derived from least-impacted reference sites (Ohio EPA 1987b, 1989a; DeShon 1995; Yoder and Rankin 1995). Fish communities were sampled using generator-powered, pulsed D.C. electrofishing units and a standardized methodology (Ohio EPA 1987b, 1989b). Fish community attributes were collectively measured with the Index of Biotic Integrity (IBI; Karr 1981; Karr et al., 1986) modified for Ohio streams and rivers (Yoder and Rankin 1995; Ohio EPA 1987b). Habitat was assessed at all fish sampling locations using the Qualitative Habitat Evaluation Index (QHEI; Rankin 1989, 1995). The QHEI is a qualitative, visual assessment of the functional aspects of stream macrohabitats (e.g., amount and type of cover, substrate quality and condition, riparian quality and width, siltation, channel morphology, etc.).

Two indicators of urbanization were developed for the Cuyahoga River basin, housing density and urban land use cover. Housing density by Census Block Group was obtained from the 1990 Census of Population (U.S. Bureau of Census,1990). Urban land use cover was derived from Landsat Thematic Mapper satellite imagery of land cover classification (September 1994) provided by the Ohio Department of Natural Resources. The number of housing units per hectare was calculated for the subwatershed upstream from each fish and macroinvertebrate sampling point to the boundary of the watershed. The percent urban land use for subwatersheds upstream from the fish sampling locations only were similarly calculated for both the Cuyahoga Basin and Columbus area study areas.

Statistical Analyses

IBI scores were regressed against chemical water quality parameters, an index of habitat quality (QHEI), and housing density. ICI scores were regressed against chemical water quality parameters and housing density. Water quality parameters were expressed as the average concentrations of phosphorus, dissolved oxygen (D.O.), nitrate+nitrite-nitrogen, ammonia-nitrogen, arsenic, lead, and cadmium (macroinvertebrates only) based on grab samples collected 6-8 times during June-October. Lead was highly intercorrelated with zinc, copper and chromium. Arsenic and cadmium were intercorrelated at fish sampling locations. Transformations used to correct departures from normality are provided in Table 1.

The relationship between different levels of urbanization, as indicated by housing density or percent urban land use (IBI only), and performance of the IBI, ICI, and selected metrics was further quantified using an analysis of variance model where quartile distributions of housing density and percent urban land use (e.g., 1st quartile ≤

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Table 1. Parameter Estimates from the Regression of IBI on Water Quality Variables, Habitat Quality (QHEI) and Housing Density, and ICI on Selected Water Quality Variables and Housing Density.

		Index of Biotic Integrity (IBI)			
N: 63	Multiple R: 0.606	Squared multiple R: 0.368			Adjusted R^2: 0.274
Effect	Coefficient	Std Error	t	P(2 Tail)	Adjusted R^2
Constant	23.318	11.019	2.116	0.039	
$Log_{10}(Ar)$	5.123	9.740	0.526	0.601	-0.011
Dissolved Oxygen	0.549	0.852	0.644	0.522	0.006
$Log_{10}(Pb)$	3.997	5.923	0.675	0.503	0.022
$1/NH_3$	-0.098	0.107	-0.916	0.364	-0.011
QHEI	0.091	0.095	0.952	0.346	0.071
$Log_{10}(TP)$	-7.876	4.781	-1.647	0.105	0.048
$Log_{10}(NO_x)$	-4.484	2.053	-2.184	0.033	0.063
(House/Hectare)	-7.171	1.769	-4.053	0.000	0.274

25th percentile of housing density, etc.) were used as factor levels. Metrics of the ICI that were used as dependent variables included the number of Ephemeroptera, Plecoptera and Trichoptera (EPT) taxa, the percent composition of mayflies, other dipterans/non-insects, and tolerant taxa. IBI metrics used included the percent composition of omnivores, tolerant fishes, sensitive fishes, and insectivores. IBI scores and metrics from a subset of samples in the Cuyahoga Basin with drainage areas less than 100 mi^2 were also analyzed according to percent urban land use in a similar manner to examine for potential differences due to stream and watershed size. Because sample sizes varied widely in the subsets, multiple comparisons were made using Sheffé's procedure (Neter et al., 1991). An analysis of covariance model was constructed for Columbus area streams using quartiles of percent urban land use as factor levels, QHEI as a covariate, and IBI scores, percent composition of tolerant fishes, insectivores, and omnivores, the number of darter and sculpin species, and number of sensitive species as dependent variables. Multiple comparisons were made using Tukey's procedure (Neter et al., 1991).

Because Cuyahoga River basin streams are subject to a variety of multiple stressors, fish sampling sites were qualitatively classified by predominant impact type and regressed against percent urban land use cover (log_{10} transformed) as a comparison to the results derived by using housing density and to determine the influence of impact type on the regression function. Impact types were defined as least impacted, estate (i.e., subwatersheds with large lot-size residential homes or green space provided by parks), sites reflecting gross instream habitat alterations (i.e., channel modifications or impoundment), sites impacted directly by discharges from combined sewer overflows (CSOs), sites impacted by wastewater treatment plant discharges alone and with CSOs, sites with evidence of impacts by legacy pollutants, or urbanization only. Regression coefficients from a subset of least-impacted, estate, and urban-only sites with drainage areas less than 100 mi^2 were compared to the same subset of sites for all drainage areas. Results of an ANOVA model using quartile distribution of percent land use as a factor level effect and IBI scores as independent variables were compared to those derived from the housing density model. Housing density, as an indicator of the degree of urbanization, was further evaluated by comparison with percent urban land use.

Housing Density and Biological Performance

When paired with chemical water quality data, housing density explained approximately 27% and 59% of the variation in IBI and ICI scores in the Cuyahoga River basin (Table 1). Of the water quality variables tested, only nitrate+nitrite-nitrogen and ammonia-nitrogen explained a small, but significant proportion of the variation in IBI and ICI scores (≈3% and 1%, respectively). For all IBI and ICI scores, housing density accounted for 31% and 23% of the variation in scores. Multiple comparisons of factor levels based on quartile distribution of housing density identified a threshold level of urbanization, coinciding with 2.53 housing units per hectare, beyond which IBI or ICI scores will increasingly fail to attain the biological criteria for the warmwater habitat use designation (Figure 1).

Shifts within the macroinvertebrate community were also associated with a threshold level of urbanization (Figure 2). The number of EPT taxa were significantly higher at the lowest levels of urbanization. Conversely, the percent composition of pollution tolerant taxa collected from the artificial substrate samplers increased sharply at sites exceeding the twenty-fifth percentile of housing density. Similarly, the percent composition of other dipterans and non-insects increased with increasing urbanization. The percent composition of mayflies found on the artificial substrates did not change with increasing level of urbanization (Figure 2).

Shifts in the compositional metrics of the fish community were associated with the degree of urbanization in the Cuyahoga River basin (Table 2) and included an increase in the relative abundance of tolerant and omnivorous fish. The relative abundance of omnivorous fishes, however, tended to be highest at intermediate levels of urbanization, but differences were not statistically significant for the subset of streams with drainage areas less than 100 mi^2. Insectivorous fishes were least abundant when housing density exceeded the seventy-fifth percentile threshold.

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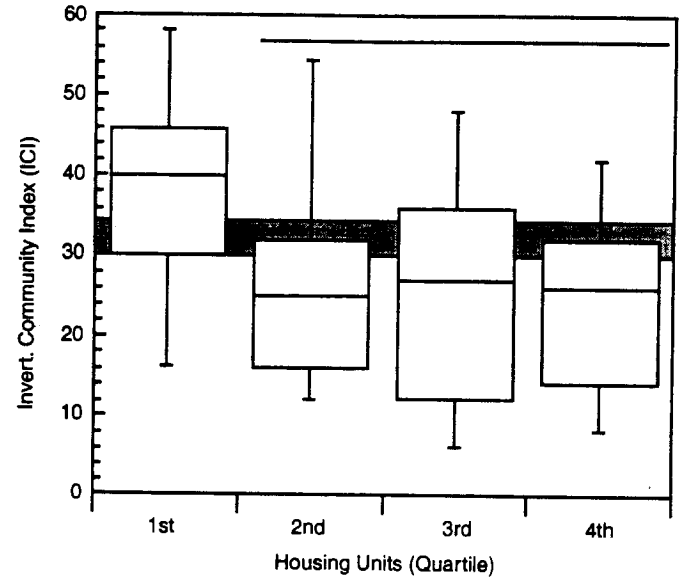

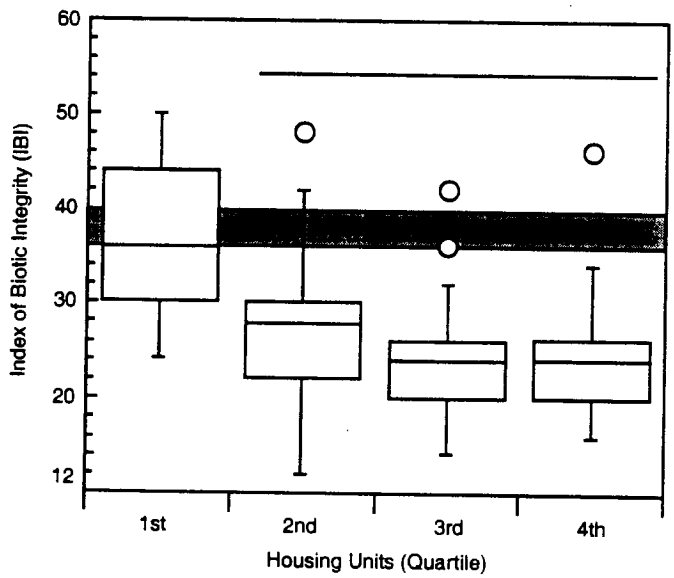

Figure 1. Distributions of Index of Biotic Integrity (IBI; lower) and Invertebrate Community Index (ICI; upper) scores from the Cuyahoga River basin plotted by quartiles of housing density upstream from sampling locations. The level of urbanization is given by quartiles of housing density per hectare of the subwatershed upstream from sampling locations. Horizontal lines spanning adjacent box plots indicate similar means. Levels of housing density per hectare corresponding to the 25th, 50th and 75th percentile are 2.53, 4.45 and 7.26 units/ha, respectively. The shaded areas indicate the applicable biological criterion and the range of insignificant departure.

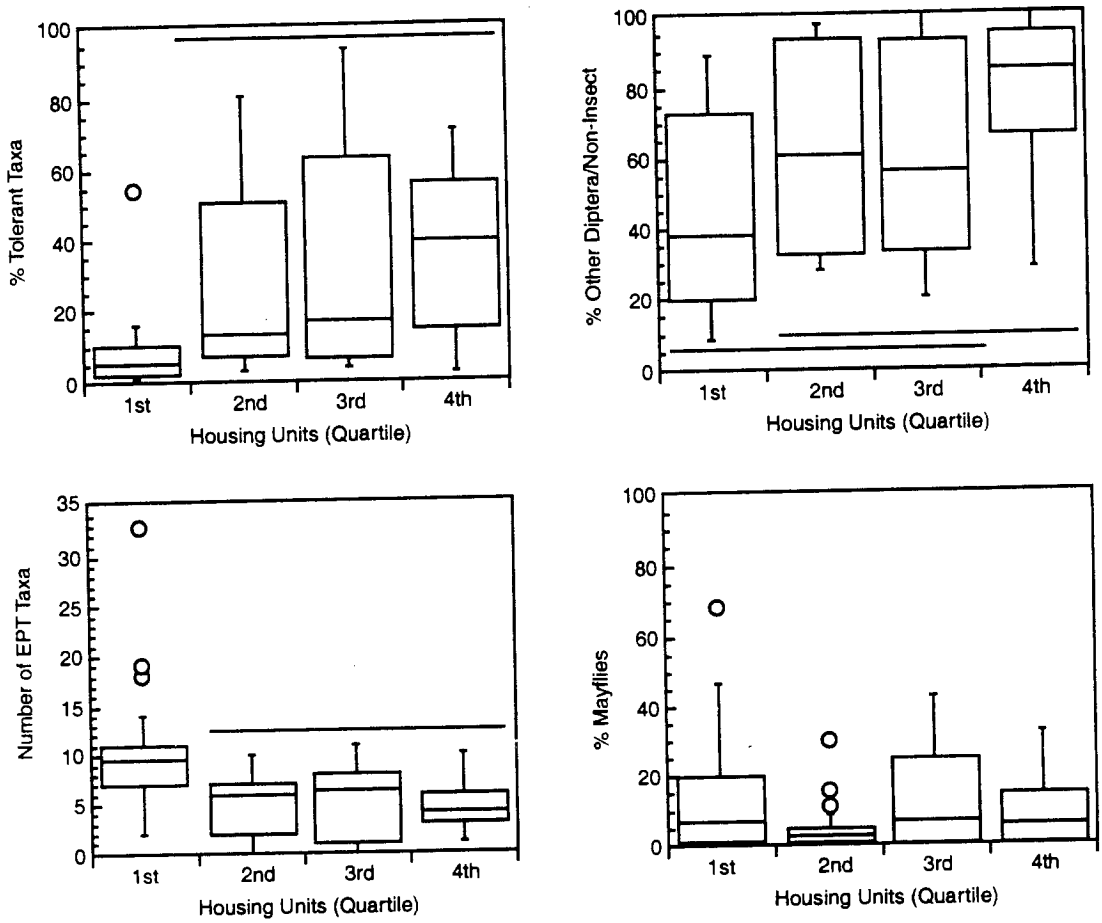

Figure 2. Performance of four Invertebrate Community Index (ICI) metrics in relation to housing density for the Cuyahoga River basin. The level of urbanization is given by quartiles of housing density per hectare of the subwatershed upstream from sampling locations. Horizontal lines spanning adjacent box plots indicate similar means. Levels of housing density per hectare corresponding to the 25th, 50th and 75th percentile are 2.53, 4.45 and 7.26 units/ha, respectively.

Urban Land Use and Biological Performance

The percentage of urban land use cover explained 26.7% of the variation in IBI scores in the Cuyahoga River basin, similar to that explained by housing density. When classified by quartile level of percent urban land use cover, the mean of IBI scores in the first quartile was significantly higher than those in the third or fourth quartile (Figure 3). However, classification by percent urban land use cover showed a more continuous decrease in mean IBI scores with an increasing level of urbanization than did housing density. Multiple comparisons of component IBI metrics classified by level of urban land use cover showed similar average responses to increasing urbanization as did classification by housing density (Table 2). However, intraquartile variation of the metric responses was greater among urban land use cover than for housing density, leading to fewer significant differences between means and reflecting the more continuous decrease in mean IBI response with respect to percent urban land use cover.

Significant differences in mean IBI scores between the levels of urban land use were also found for Columbus area streams (Figure 3). Mean IBI scores from streams with less than 3% urban land use were significantly higher than those with greater than 33% urban land use (Figure 3). Shifts in the composition of the fish community associated with increasing percent urbanization included the loss of darters, sculpins, and other pollution and habitat sensitive species, decreased abundance of insectivores, and an increase in the proportion of tolerant fishes (Table 3).

Discussion

Threshold levels of urbanization beyond which biological communities are likely to be impaired have previously been identified in the range of 8% to 20% impervious cover within a watershed (Schuler 1994). The threshold levels in our study of approximately 8% and 33% urban land use cover for the Cuyahoga River basin and Columbus area streams, as identified by analysis of variance, is in general agreement with the studies reviewed by Schuler (1994).

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Table 2. Factor Level Means and Sheffe' Groupings for Selected Fish Community IBI Metrics Sampled in the Cuyahoga River basin in Relation to Urban Land Use Indicators. Means Sharing a Common Letter are Not Significantly Different. The Asterisks Denote where Significant Differences Between Groups were Not Detected in Multiple Comparisons for the Percent Tolerant Group from all Sites, and for the Number of Sensitive Species in Streams Less than 100 Mi², The Overall F Tests Indicated a Significant ($P < 0.05$) Linear Relationship.

Urban Indicator (Quartile)	N	Number of Sensitive Species		Percent as Insectivores		Percent as Tolerant		Percent as Omnivores	
				All sites - Housing Units per Hectare					
1st	22	A	3.0	A	49.9	A	31.9	A	15.1
2nd	21	AB	2.1	A	39.2	AB	38.2	B	28.3
3rd	19	CB	1.4	A	27.4	AB	48.1	B	48.4
4th	21	C	0.4		10.5	B	71.4	AB	22.1
				All sites - Percent Urban Land Use					
1st	22	A	2.5	A	49.5	A*	35.9	A	18.7
2nd	21	A	2.2	A	41.4	A	40.4	A	27.7
3rd	19	AB	1.4	B	18.6	A	54.2	A	38.7
4th	21	B	0.7	B	14.6	A	58.2	A	31.0
				Drainage Area < 100 mi² - Percent Urban Land Use					
1st	12	A*	2.5	A	44.8	A	46.5	A	22.7
2nd	11	A	2.0	A	40.8	A	44.3	A	20.4
3rd	9	A	0.8	B	13.2	A	66.1	A	11.5
4th	17	A	0.6	B	10.5	A	69.7	A	24.9

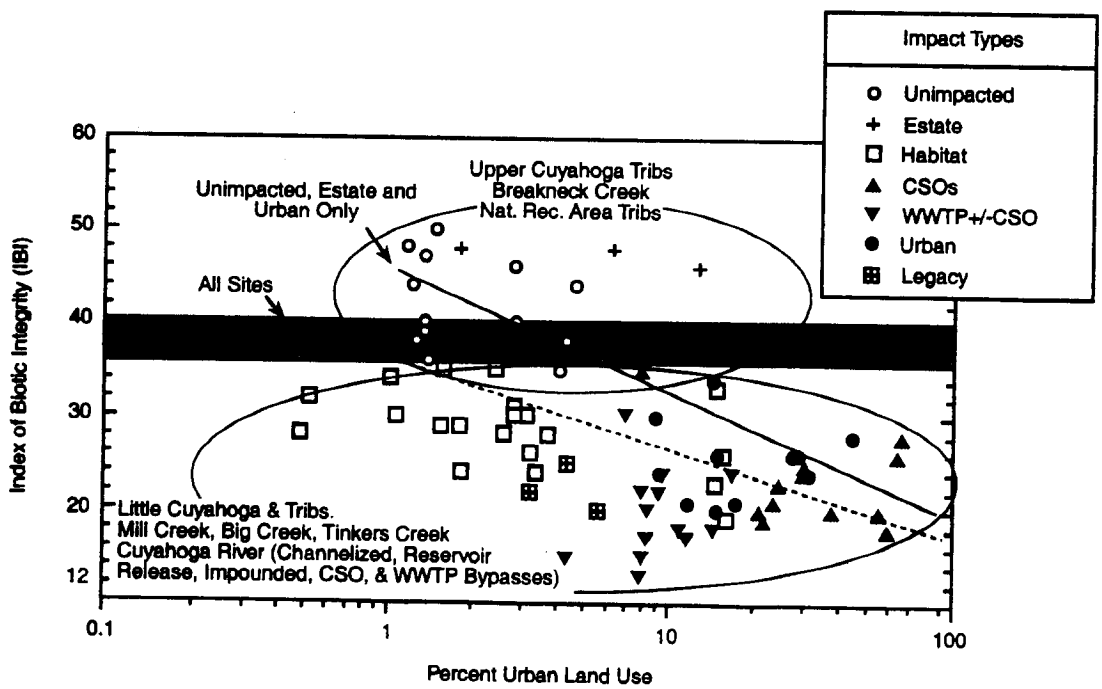

Figure 3. Distribution of Index of Biotic Integrity (IBI) values plotted by quartiles of percent urban land use cover upstream from sampling locations for all sites in the Cuyahoga River basin, Cuyahoga basin sites with drainage areas less than 100 mi², and Columbus area streams. The shaded areas indicate the applicable biological criterion and the range of insignificant departure.

Table 3. Factor Level Means and Tukey Groupings for Selected Components of Fish Communities Sampled in Columbus Area Streams. Means Sharing a Common Letter are not Significantly Different. Samples Sizes in Ascending Order by Quartile are 20, 18, 20 and 20; Two Cases in the Second Quartile had Missing Values Due to No Fish.

Percent Urban by Quartile	Number of Sensitive Species		Number of Darters/Sculpins		Percent as Insectivores		Percent as Tolerant		Percent as Omnivores	
1st	A	3.3	A	3.1	A	40.8	A	54.6	A	13.0
2nd	A	3.8	AB	2.2	AB	32.0	A	56.3	A	13.9
3rd	B	0.9	CB	1.1	B	17.0	B	82.7	A	10.5
4th	B	0.6	C	0.6	B	19.7	B	78.3	A	5.1

However, the threshold level identified by regression for the Cuyahoga River basin was influenced by the presence of other stressors (e.g., CSOs, point sources, legacy pollutants). The elimination of those sites impacted by other stressors from the regression resulted in an increased threshold of urbanization (Figure 4). Although other stressors acted as covariates in a sense, these were not amenable to an analysis of covariance because each occurred in relatively discrete groupings along the continuum of increasing urbanization. Analysis of variance was better able to identify a threshold level by contrasting discrete ranges (i.e., quartiles) along the entire range of increasing urbanization (Figure 3).

Similar patterns in the effect of increasing urbanization on biological communities were evident for both the Cuyahoga River basin and Columbus area streams. Detectible differences in the number of sensitive fish species in Columbus area streams occurred at lower levels of urbanization than did IBI scores, illustrating the role of sensitive species as sentinels of urban effects. Sensitive fishes are rare in the Cuyahoga River basin as a whole due to historic, complex, and widespread anthropogenic stressors, yielding less response and higher variation associated with interquartile means compared to the Columbus area streams. However, the number of EPT taxa, a sensitive macroinvertebrate guild, similarly acted as sentinels of urbanization given that EPT abundance was significantly reduced at relatively low levels of urbanization. The abundance of mayflies, showing little correlation with the level of urbanization, did not respond in a manner similar to the number of EPT taxa. While this may reflect the difference in collection technique as percent mayflies are based on the data from artificial substrates, whereas EPT taxa are based on data collected from natural substrates, it may also be due to differing sensitivities within the EPT guild. This result, in combination with the response of the fish community, implies that substrate degradation is a major factor which limits aquatic communities at relatively low levels of urbanization.

The relative abundance of omnivores tended to be highest at intermediate levels of urbanization when all sites in the Cuyahoga Basin were included. This response was due in part to enrichment by wastewater treatment plant discharges and CSOs discharging to the Cuyahoga River mainstem. No differences were detected for the subset of streams with drainage areas less than 100 mi^2, nor in the Columbus area streams. However, the relative abundance of insectivores was negatively correlated with increasing urbanization in both study areas, suggesting a disruption within the aquatic food web. Conversely, the proportion of tolerant fishes was positively correlated with increasing urbanization. The high proportion of tolerant fishes at the highest levels of urbanization is indicative of both degraded habitat and water quality, specifically toxicity and organic enrichment. Collectively, these changes in biological communities suggest a continuous negative response to increasing urbanization starting with the loss of sensitive fish and macroinvertebrate species at comparatively low levels of urban development (<5% urban land use) due to substrate degradation, disruption within the aquatic food web at intermediate levels of development, and a response to toxicity, organic enrichment, or both at higher levels of development (>15% urban land use).

Overlaying impact types with percent urban land use (Figure 4) demonstrates that the negative effects of urbanization and associated cofactors (e.g., imperviousness, polluted runoff, altered hydrology) may be partially offset by beneficial land use practices. Biological performance at sites impacted by estate-type residential developments remained comparatively intact and attained the ecoregion biocriteria even at relatively high levels of urbanization (up to 15%). The best performing sites within those watersheds also had relatively intact stream habitat and well-vegetated, wider riparian buffers. Conversely, sites with increasingly modified habitats performed poorly and failed to attain the biocriteria regardless of the degree of urbanization. The most degraded sites were associated with either poorly treated sewage, CSOs, and/or a high degree of urbanization. These findings agree with those of Steedman (1988) who demonstrated a co-relationship between riparian zone quality and land use in terms of how each affected the fish communities of Toronto area streams. Horner et al. (1997) found the steepest rates of decline in biological functioning (in terms of the B-IBI; Kerans and Karr 1992) to occur with increases in impervious cover of as little as 1-6% in streams flowing into Puget Sound, Washington. Exceptions occurred where urban land use was mitigated by extensive riparian protection or other management interventions, but these factors ceased to be effective above 45% as impervious land cover.

Unlike the Cuyahoga River basin, the Columbus area streams were not subject to extensive CSO impacts and

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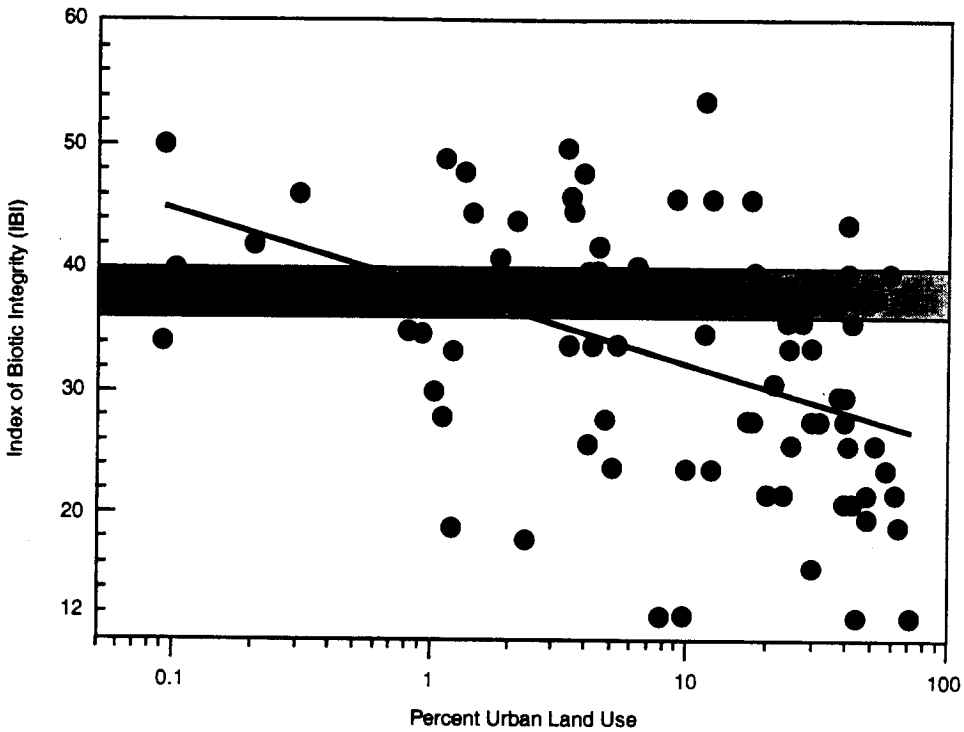

Figure 4. Index of Biotic Integrity (IBI) scores from sites sampled in the Cuyahoga River basin plotted by stressor group (symbols) agains percent of urban landuse for sites draining less than 100 mi². The fitted regression lines are for all points and those lacking stressors other than urbanization. The shaded areas indicate the applicable biological criterion and the range of insignificant departure.

industrial legacy pollutants were virtually absent. Consequently, the threshold level of urbanization precluding attainment of the biological criteria was higher for the Columbus area streams (Figure 5), results which are analogous to that for sites influenced by the estate impact type in the Cuyahoga River basin. In fact there were a few sites with urban land use as high as 50% which fully attained the ecoregional biocriterion. This suggests that the type of urban development strongly influences the attainability of aquatic life uses within a watershed. Furthermore, factors such as impermeability and urbanization alone do not automatically disqualify streams from meeting designated uses based on biological criteria.

Although housing density and percent urban land use demonstrated a strong linear relationship (Figure 6), each urban indicator showed somewhat differing results. The percent of urban land use indicator, which is a more precise measure of urbanization and imperviousness, was negatively correlated with biological community performance. By comparison, the housing density indicator showed a discrete threshold between the lowest quartile and all others. The principal difference is that high-quality sites were more frequently associated with the second quartile of percent urban land use than for housing density, reflecting good IBI scores from relatively urbanized subwatersheds containing large residential lot sizes and more green space. Also, urban land use within successive quartiles of housing density apparently becomes increasingly mixed as inferred by increasing interquartile variation in percent urban land use (Figure 6). Higher levels of housing density coincided with increased industrial, commercial, and transportation related land uses. The difference in results by urban indicator underscores the importance of maintaining natural features within a watershed including instream habitat, vegetated riparian buffers of adequate width, and green space in addition to minimizing and controlling chemical impacts from wastewater treatment plants, CSOs, and other sources.

Implications for Use Attainability

Uses designated for specific water bodies are done so with the expectation that the criteria associated with the use are reasonably attainable. If CWA goal uses (e.g., warmwater habitat in Ohio) are found to be unattainable, lower uses may be established and assigned on a case-by-case basis. Federal water quality regulations (40CFR Part 131.10[g]) generally specify three criteria for setting designated uses below "fishable/swimmable" standards as follows: 1) imposition of the criteria for a higher use would result in widespread, adverse socioeconomic impacts; 2) the criteria are not attainable due to natural background conditions; or 3) the criteria are not attainable due to irretrievable, anthropogenic impacts.

Compliance with the aquatic life uses defined in the Ohio WQS are determined primarily by the biological criteria (OAC 3745-1-07) which are stratified according to designated use, ecoregion, and stream size. As such this repre-

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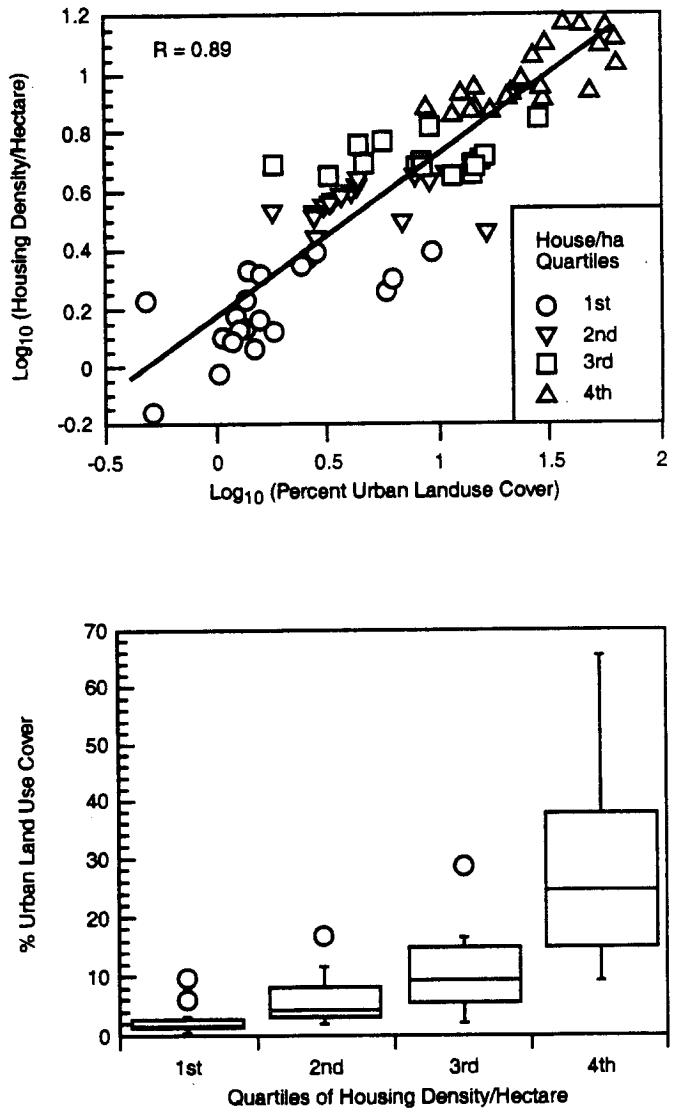

Figure 5. Index of Biotic Integrity (IBI) scores from sites sampled in Columbus area streams against percent of urban land use. The fitted regression lines are for all points and those lacking stressors other than urbanization. The shaded areas indicate the applicable biological criterion and the range of insignificant departure.

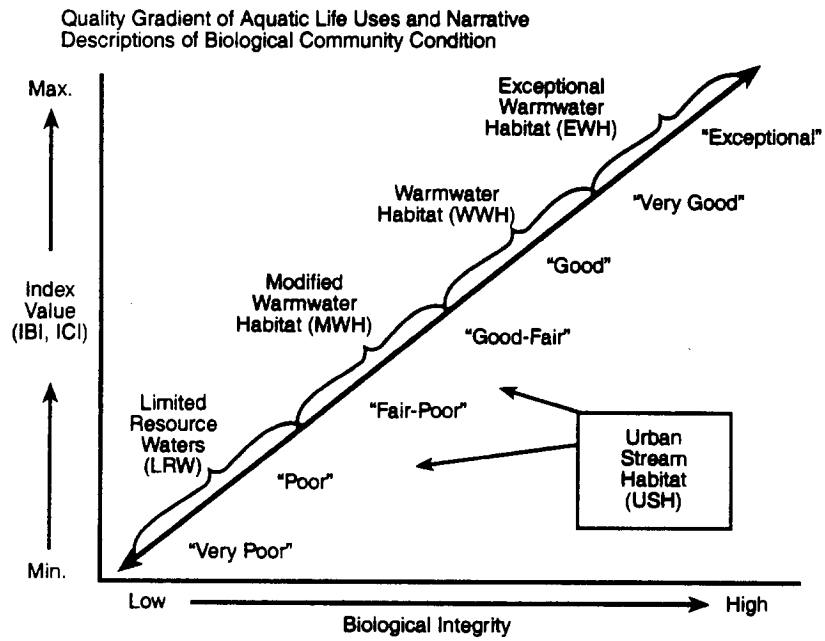

Figure 6. Relationship between housing density and percent of urban land use cover for subwatersheds upstream from fish sampling locations in the Cuyahoga River basin. The upper plot shows housing density as a function of percent urban landuse cover. The lower plot shows distributions of percent urban land use cover within quartile levels of housing density per hectare.

sents a stratified system of uses and criteria that occur along a gradient of biological integrity as expressed by the biological indices which comprise the numerical biological criteria (Figure 7). For most Ohio streams the "default" expectation is attainment of the warmwater habitat (WWH) use provided the physical habitat is relatively intact and no extensive alterations are evident. Obvious anthropogenic alterations to small urban streams such as culverting, relocation, bank and channel stabilization with artificial structures, and extensive channelization are relatively easy to identify and assess. In such cases, the Limited Resource Waters (LRW) use designation is assigned which means that the minimum level of protection (i.e., prevention of lethality) afforded by the Ohio WQS applies. The difficulty is with small urban streams that exhibit adequate habitat (as defined by the QHEI score), but which fail to attain the WWH biocriteria. The recent finding that no urban headwater stream sites in the Ohio EPA database attain the WWH biocriteria (Yoder and Rankin 1997) only serves to further the notion that the degree of watershed urbanization can preclude the WWH use regardless of the site-specific habitat quality.

Recently, the imperviousness of the watershed has been used as an indicator which is correlated with use attainability. If the frequently cited threshold of 25% impermeability is used, streams in watersheds with greater than this value would be unlikely to ever attain a beneficial use regardless of site and reach factors. The results of our study suggest that there is a threshold of watershed urbanization beyond which attainment of the WWH use is increasingly unlikely. However, this threshold is different among watersheds as evidenced by the results from the Cuyahoga Basin and Columbus area streams. Co-occurring factors such as pollutant loadings, watershed development history, chemical stressors, and watershed scale influences such as the quality of the riparian buffer and the mosaic of different types of land use also greatly influence the biological quality in the receiving streams.

While the development of indicators of watershed urbanization has merit from a management and decision-making standpoint, there are simply too many other factors, some of which are controllable and amenable to remediation, to use it as a sole determinant for aquatic life attainability. We suggest that the co-factors in addition to urban watershed indicators be better developed and tested using datasets from broader geographic areas and spanning the extremes of the urbanization gradient. One goal should be to develop, if appropriate, an urban stream habitat designation that would fit along the already existing hierarchy of aquatic life use designations in Ohio (Figure 7). We have indicated on Figure 7 where the biological criteria for this potential new designation might occur compared to the already existing hierarchy of aquatic life uses in the Ohio WQS. However, placing it on the existing quality gradient will require substantial calibration and validation with existing datasets. Having this use would satisfy the desire to afford streams with the maximum protection practicable, while recognizing the inherent limitations that urbanization imposes on stream quality.

In the meantime, simplistic regulatory and management approaches should be limited, particularly in those water-

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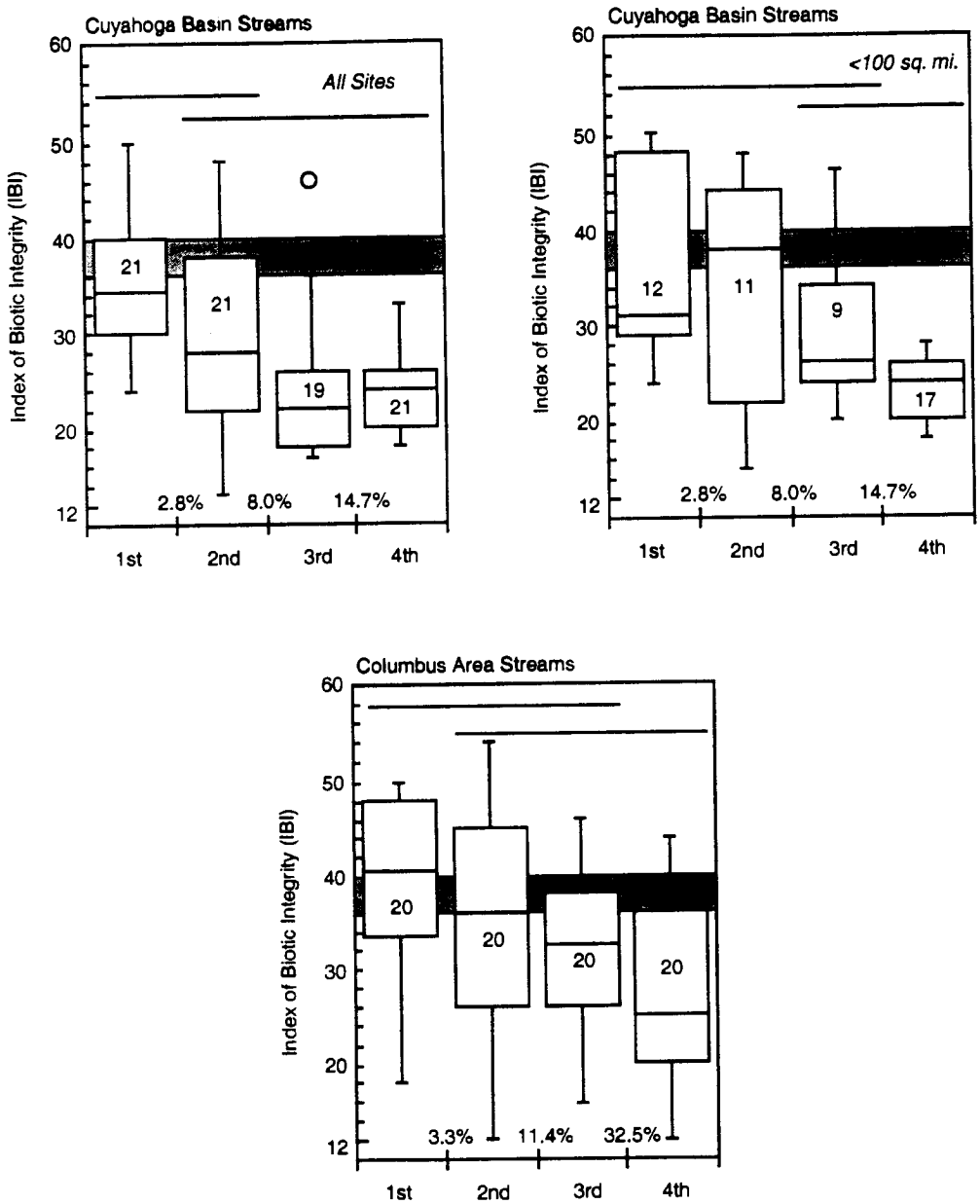

Figure 7. Index of Biotic Integrity (IBI) scores from sites sampled in the Cuyahoga River basin plotted by stressor group (symbols) against percent of urban land use for sites draining less than 100 mi². The fitted regression lines are for all points and those lacking stressors other than urbanization. The shaded areas indicate the applicable biological criterion and the range of insignificant departure.

sheds where uncertainty about the attainability of CWA goal uses (i.e., WWH and higher) exists. For example, initial approaches such as the nine minimum controls for CSOs seem reasonable. However, proceeding beyond these requirements with long-term control plans should be done cautiously and with the aid of sufficiently robust before-and-after biological and water quality assessments.

The results of our study also point out the benefits of a regular, sustained, and robust state monitoring and assessment effort (see also Yoder and Rankin 1998). Dealing with complex water quality management issues such as CSOs, stormwater, and TMDLs in urban watersheds would be difficult at best within the confines of the traditional administrative approach to water quality management. Steedman (1988) described multimetric biological indices like the IBI and ICI as being based on simple, definable ecological relationships which is quantitative as an ordinal, if not linear, measure and which responds in an intuitively correct manner to known environmental gradients. Further, when incorporated with mapping, monitoring, and modeling information, such an approach has been shown to be valuable in determining management and restoration requirements for warmwater streams (Steedman 1988; Bennet et al., 1993). The value added by a robust bioassessment and tiered use designation framework coupled with sufficiently detailed and accurate GIS information was amply demonstrated herein.

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Tampa Bay Environmental Monitoring Program

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Significant damage to, and loss of natural habitats in Tampa Bay can be traced to the uncontrolled development and pollution that started in the 1950s. Although great strides have been made over the last decade to reverse this trend, many agreed that a bay management and restoration plan, including monitoring programs that would evaluate bay conditions and progress, were needed.

The nomination and designation of the Tampa Bay National Estuary Program (TBNEP) in 1990, provided the platform to assist the community in developing a comprehensive plan to protect and restore the bay.

The process for developing the master plan includes the following components: identify and rank priority problems; assess bay conditions and needs; establish specific goals for the bay; develop management options; prepare the Comprehensive Conservation and Management Plan (CCMP); develop an implementing agreement among bay partners; implement the plan; and monitor progress (TBNEP 1996).

Methodologies for bay assessment, goal setting and development of comprehensive monitoring strategies are described. These methodologies could be useful to others interested in evaluating environmental protection and restoration schemes for natural resources.

Program Organization and Goal Setting

Since there already existed strong local and regional involvement in bay management, the TBNEP built on this commitment through the creation of its governing structure which consisted of the following committees: Policy, Management, Technical Advisory (TAC) and Community Advisory (CAC). From the Program's inception, the overall management goal was to protect and enhance the bay's natural resources. In support of this goal, the committees were required to characterize the natural systems of the bay and the impacts to these systems and define and implement actions to address those impacts. A Management Conference, with participation from all committees and stakeholders, was convened to identify priority bay issues. The following priority problems were identified by the Policy Committee in March 1991(TBNEP 1996):

- Water quality deterioration/eutrophication
- Reduction/alteration of living resources
- Lack of community awareness
- Increased user conflicts and impacts from various recreational activities, industrial and navigation needs, and urban development
- Lack of agency coordination and response
- Lack of circulation and flushing
- Hazardous/toxic contamination

Traditionally, monitoring and evaluating water quality conditions within a watershed are used to measure watershed health and productivity. Vigorous bay monitoring and management activities were being conducted by the time the TBNEP began in 1991; however, many of these activities focused on individual components and/or processes of the bay (Greening 1998). It was necessary to organize the information, coordinate bay managers' and stakeholders' participation and evaluate how these individual activities could be integrated to establish bay ecosystem management.

Quantifiable Restoration and Protection Goals

In keeping with the overall goal of protecting and restoring the bay's natural resources, the TAC worked to define species or biological communities which could be used as "indicators" of functioning bay ecosystems. The significant loss of submerged aquatic vegetative ("seagrass") habitat stood out as the premier concern of bay managers, scientists and concerned public. This habitat is crucial for many invertebrates and fish and provides for sediment stabilization (Busby and Virstein 1993). If quantifiable seagrass restoration goals and management strategies could be developed and implemented, it would be feasible to develop similar procedures for restoring other targeted habitats and natural resources.

Quantitative targets for the restoration and protection of seagrass habitat, as well as emergent habitats, were ap-

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proved at a Management Conference in 1993. The approach to habitat restoration and protection was as follows (Janicki et al., 1994):

1. Map the historic living resource distribution during a benchmark time period.
2. Map the existing distribution of these living resources.
3. Overlay the historical and existing distributions to define potential restoration and protection targets.
4. Subtract physically altered (non-restorable) areas to identify restoration targets.

Seagrass Restoration and Protection Goals

Utilizing the approach described above, it was determined that the benchmark for establishing seagrass protection and restoration goals would be the period circa 1950. This era was chosen because the area was beginning to experience explosive growth and the major development alterations were not yet complete. Additionally, comparable habitat data were not available before 1950 (NUS Corp. 1986).

Using aerial photography coupled with the Arc/Info GIS system, it was determined that the extent of seagrass coverage in 1950 (not including areas that were irrevocably altered by 1990) was estimated to be 40,400 acres (NUS Corp. 1986). In 1990, Ries (1993) estimated the seagrass habitat coverage to be approximately 25,200 acres. Having already factored the physical losses due to dredge and fill activities, the remaining losses were most likely caused by degraded water quality conditions (Janicki et al., 1994). Recent investigations suggest that the loss of seagrass meadows can be attributed to lack of sufficient sunlight because of attenuation by excess phytoplankton, suspended solids and epiphytic algal growth (Morris and Tomasko 1993; Tomasko 1993; and Stevenson et al., 1993). Excessive algal concentrations or eutrophic conditions are predominantly caused by excessive nutrient (e.g., nitrogen and phosphorous) loading.

Acreage goals for seagrass restoration and protection were developed by overlaying the 1950, 1990 and non-restorable acreage data sets. Seagrass areas observed in 1990 were designated as seagrass protection areas. All areas in which seagrasses were mapped in 1950, but which did not support seagrass in 1990 and were not classified as non-restorable, were identified as seagrass restoration areas (Greening 1998). Based on a review of the data sources, method evaluation and uncertainty in estimating the 1950 coverage, the Management Committee agreed to adopt a minimum seagrass restoration goal of 38,000 acres bay-wide. This goal includes protection of an existing 25,650 acres and restoration of 12,350 additional acres.

Development of Intermediate Targets

Assessing bay management success via living resource goals is considerably more difficult than using traditional water quality criteria because it takes much longer to realize results. It is not too difficult to evaluate annual water quality trend response to management actions. It has been demonstrated, however, that seagrass quality and quantity improvements may not be observed for decades after a management action is implemented (Johansson and Ries 1997). To ensure that correct management actions were being implemented and bay water quality improvements would lead to the achievement of the seagrass restoration and protection goal, it was necessary to establish intermediate targets so that more timely evaluations and management adjustments could be made if necessary.

In the Tampa Bay area it has been demonstrated that seagrass health and distribution are adversely affected by incident sunlight being attenuated within the water column by elevated suspended solids or phytoplankton concentrations (Lewis et al., 1985; Lewis et al., 1991). If seagrass does not receive adequate light, plant maintenance and reproduction are inhibited (Janicki et al., 1994).

For the purpose of determining the relationship between nutrient loadings to the bay and adequate water quality to support the seagrass restoration target, a two-pronged modeling approach was developed. The first was a series of empirical regression-based models to estimate external nutrient loadings consistent with the proposed seagrass enhancements (Janicki and Wade 1996), and the second was a WASP-based box model which provided a process-oriented examination of relationships between nutrient loadings, chlorophyll *a* concentration and light attenuation (Martin et al., 1996; Morrison et al., 1997).

Both the empirical and mechanistic models produced similar results, suggesting that acceptable nutrient management targets could be developed. The critical relationships that were established were external nitrogen (limiting nutrient) loads and resulting chlorophyll *a* concentrations; chlorophyll *a* concentrations and density of phytoplankton in the water column; and chlorophyll *a* concentrations and light levels at the deep edges of historic seagrass beds.

Since the estuary is about 1,031 km^2 (398 mi^2) with varying land uses, fresh water inflow, nutrient loadings and circulation patterns, it was decided that the best way to manage this system was to partition or segment according to similar conditions. The segmentation scheme defined by Lewis and Whitman (1985) was adopted to establish the official management subdivisions of the bay (Figure 1).

Following numerous scientific workshops, the TAC and Management Committee adopted chlorophyll *a* targets necessary to maintain water clarity needed for seagrass growth for each bay segment. The adopted segment-specific annual average chlorophyll *a* targets (8.5 *ug*/l for Old Tampa Bay; 12.3 *ug*/l for Hillsborough Bay; 7.4 *ug*/l for Middle Tampa Bay; and 4.6 *ug*/l for Lower Tampa Bay) will be used as indicators for evaluating water quality conditions necessary to meet long-term seagrass restoration and protection goals.

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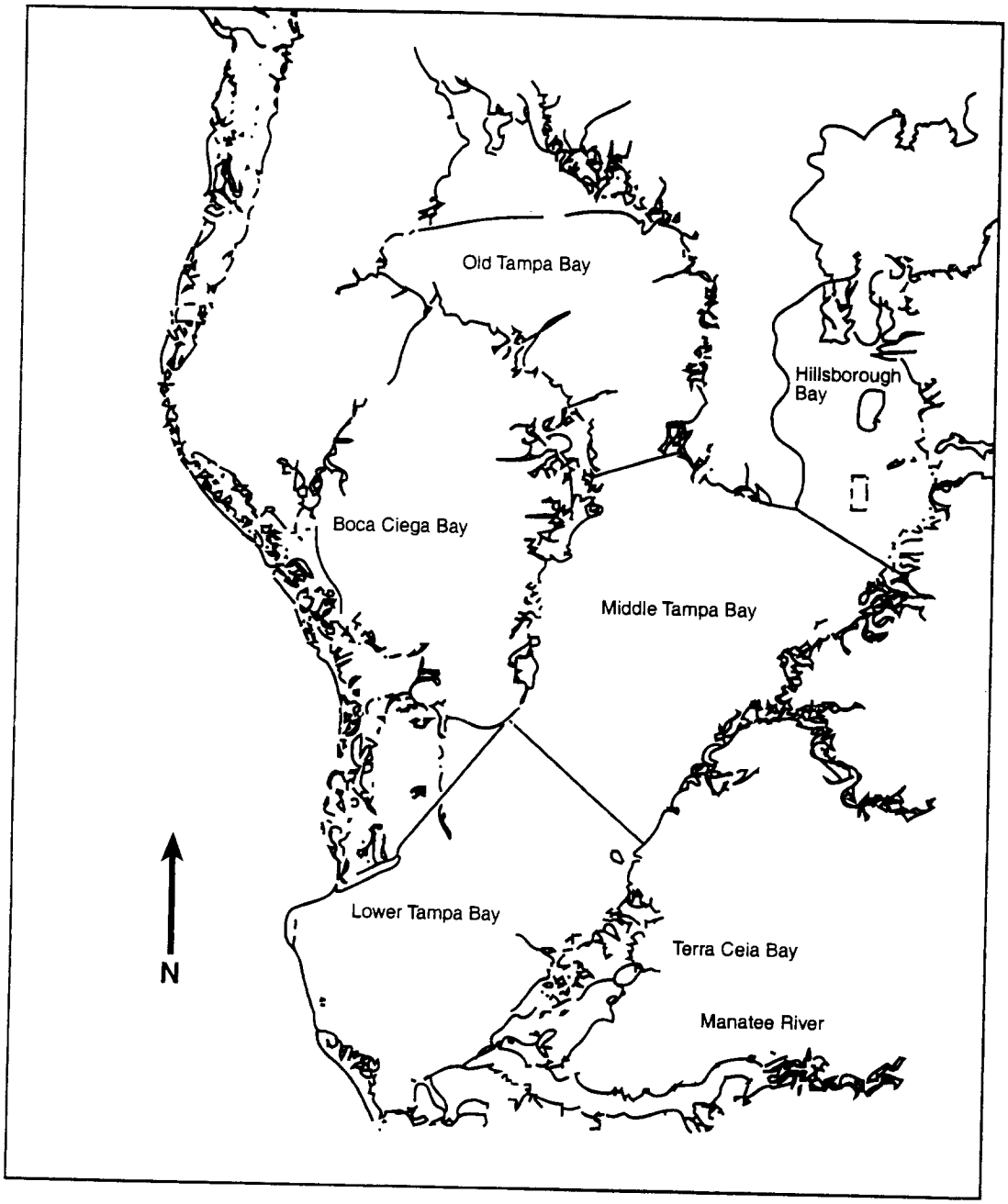

Figure 1. Tampa Bay, Florida segmentation scheme.

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Nitrogen Management Strategy

Based on light conditions observed during a present day period (1992-1994), it was determined that water quality conditions were adequate to support the long-term seagrass restoration goals; therefore, a nitrogen loading "hold-the-line" strategy was adopted (Janicki and Wade 1996). This means that if the nitrogen loads observed during the period 1992-1994 remained constant into the future, it would be possible to achieve the seagrass restoration goal. However, it is estimated that by the year 2010, the watershed will experience a 20% increase in population and approximately a 7% increase in annual nitrogen loading (Zarbock et al., 1996).

In lieu of developing stringent future nitrogen load reduction allocations, local governments and agency partners in the TBNEP developed an unprecedented interlocal agreement (Memorandum of Understanding for the Federal agencies) pledging the development and implementation of action plans that will defer or reduce future nitrogen loadings, thereby maintaining the "hold-the-line" commitment.

Monitoring and Reporting

The process for developing monitoring strategies for this program was as unique as that used in developing the living resource goals. There were many monitoring activities ongoing when the TBNEP program was established, but these activities were localized and designed for specific needs.

The first task was to evaluate all of the different monitoring programs being conducted for Tampa Bay to determine whether they would meet the monitoring criteria for National Estuary Programs (USEPA 1991). Their criteria include: "measuring the effectiveness of management actions and programs implemented under the CCMP and providing essential information that can be used to redirect and refocus the management plan." Additionally, a 1992 monitoring workshop recommended four additional monitoring objectives (Versar 1992):

- To estimate the areal extent, and temporal trends in areal extent, of habitat conditions in Tampa Bay not meeting living resource requirements
- To assess the relative abundance and condition of fish populations of Tampa Bay over time
- To estimate the areal extent and quality of seagrass, mangroves, and coastal marshes in Tampa Bay over time and
- To estimate the areal extent and trends in areal extent of oligohaline habitat in Tampa Bay and its tributaries

To accomplish most of these monitoring objectives, it was decided that a probability-based sampling design be developed that would allow statistically valid, unbiased estimates of abundance and areal extent of key indicator species on a bay-segment and bay-wide basis. The chosen design was based on the U.S. Environmental Protection Agency's (USEPA) Environmental Monitoring and Assessment Program (EMAP) (Versar 1992). Since most of the existing monitoring activities were biased, fixed station designs, modifications to these programs were necessary.

In order to prepare and implement the new monitoring strategies, the local and regional agencies responsible for sample analyses and data reporting created a coalition known as the Florida West Coast Regional Ambient Monitoring Program ("RAMP"). RAMP participants meet regularly for the purpose of standardizing methodologies, evaluating quality assurance between laboratories, and coordinating field sampling strategies. These coordinated activities have 1) allowed the local agencies to develop expertise in areas other than general water quality monitoring (e.g., benthic and seagrass monitoring); 2) economized resources by linking bay areas and programs instead of creating overlap; and 3) allowed utilization of the existing EMAP probabilistic design to build monitoring programs required by other regulations (i.e., NPDES stormwater).

Another very important component of the monitoring strategy is reporting. The monitoring design described has both short- and long-term targets and goals. In order to provide bay resource managers timely information, the TBNEP, with assistance from state, regional and local scientists conducting monitoring and research, will prepare a biennial Tampa Bay Environmental Monitoring Report. The information provided in these reports is intended to provide decision makers timely access to information critical for successful restoration and protection of Tampa Bay's living resources.

Conclusions

The restoration and protection strategies designed for Tampa Bay by local, regional, state and federal participants epitomize coordinated ecosystem management. The development of resource-based targets, as defined by the environmental requirements of critical living resources, is difficult but essential for maintaining the health and productivity of critical habitats.

The real key to successes experienced in Tampa Bay is the concerted effort put forth by agency personnel, elected officials and concerned members of the public in dealing with difficult, complex issues and making critical management decisions. These accomplishments were possible because participants possessed dedication and commitment to restoring and protecting the living resources of Tampa Bay.

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Retrofit Opportunities for Urban Waters Using Soil Bioengineering

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Abstract

Soil bioengineering retrofits have been used to meet specific objectives, such as flood control, stormwater management, bank stabilization, aesthetics and habitat enhancement. In response to increasing environmental concerns, soil bioengineering systems using woody vegetation provide streambank protection in urban waters while maximizing ecological and water quality benefits in urban waters. Retrofit opportunities to restore incised and enlarged channels as well as those that have been relocated and/or straightened are discussed in this paper.

Case studies are presented to illustrate the use of this technology on several projects where geotechnical, hydrologic/hydraulic, and environmental objectives needed to be met. These included a flood control stream in an urban linear park in Charlotte, North Carolina; a relocated stream in Portland, Oregon; a flood control channel through a residential neighborhood in Houston, Texas; and an erosion and flood control stream restoration project in a residential neighborhood in Wilmington, North Carolina. Information is presented for evaluating alternative soil bioengineering streambank protection measures and selecting those that best achieve the desired goals

Introduction

Nonpoint source impacts on urban streams and adjacent lands have become increasingly damaging in many U.S. metropolitan areas, especially where streams have been straightened or relocated. Straightening of urban streams is often done to consolidate land for development, or to bypass lakes or public parklands. Uncontrolled stormwater runoff is also a significant contributor to the erosion of channel beds and banks of urban streams. The increased frequency and magnitude of these flows and their associated velocities results in stream damage and loss of valuable land.

Stream power is proportional to the product of discharge and slope. Both variables are increased when streams are straightened during development and subjected to increased discharges. Stormwater detention is a relatively recent concept, and most of the subdivisions and commercial sites developed during the 1960s and 1970s have no stormwater detention facilities (Nunnally and Sotir 1997).

Traditional land "repackaging" for convenience and short-term resale ignore not only the sediment transport that occurs during the construction phase and long-term channel stabilization problems; they also ignore water quality, aesthetic values and a host of environmental benefits including aquatic and terrestrial habitat. The opportunity of retrofit recovery through the use of soil bioengineering is discussed in this paper using case studies.

Geotechnical, Hydraulic and Hydrologic Benefits of Soil Bioengineering

Soil bioengineering methods offer a broad range of mechanical benefits when installed as retrofits to damaged urban stream systems (Table 1). Geotechnically, they offer immediate soil reinforcement up to a depth of 12 feet. The use of brushlayers with natural or synthetic geogrids, is especially useful where space is constrained, as these methods may be constructed on very steep slopes (1H to 1V; in some cases as steep as .25H to 1V). Installed vegetation offers many hydrologic values in that the embedded branches serve as horizontal drains converting parallel seepage flow to vertical flow, thus offering improved overall slope stability. Surface protection and reinforcement is further increased when live branches develop roots and top growth. The roots tend to consolidate the soil particles by reinforcing the soil mantle, reducing the possibility of slips and displacements. The top leaf and branch growth provides direct bank protection and reduces velocities while redirecting the flow away from the bank.

Environmental Benefits of Soil Bioengineering

Soil bioengineering offers a variety of environmentally sound retrofit opportunities for urban waterways. The main benefits of different methods have been summarized in Table 2. They may serve as a useful guide in the selection of specific vegetative methods. Soil bioengineering systems that use live siltation construction or create scour

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Table 1. Soil Bioengineering System Geotechnical and Hydraulic Goals and Benefits

Goals and Benefits	*Live Stakes	Live Fascine	Live Siltation Construction	Branch-packing	Brush-mattress	Live Cribwalls	Vegetated Geogrid	Live Boom(s)
	Soil Bioengineering Methods							
Geotechnical	fair	good to very good	n/a	good	n/a	excellent	excellent	n/a
Hydraulic	fair to good	good to very good	very good to excellent	excellent	very good	good to very good	good	fair to good

*After established (1 year).

Table 2. Environmental Benefits of Soil Bioengineering Streambank Restoration Systems

Goals and Benefits	Vegetated Geogrid	Live Cribwall	Live Boom	Live Siltation Construction	Brush-mattress	Live Fascine	Live Stake
	Soil Bioengineering Methods						
Shade and	excellent	excellent	very good	excellent	good to very good	good	fair to good
Create or Preserve Scour	good	very good	excellent	n/a	n/a	n/a	n/a
Riparian Habitat	fair to good	fair to good	n/a	very good to excellent	excellent	good to very good	fair to good
*Recreation	very good	very good	n/a	good to very good	good to very good	good	fair

*Visual perspective

holes using live booms (dikes composed of woody vegetation and soil) are excellent choices as part of a bank protection system where shade and overhanging cover or pool habitat is desirable (Sotir 1997c). The habitat for mammals and birds will improve over time in such areas for nesting, migration and cover.

In addition to the selection and orientation of methods, the choice of woody vegetation species in soil bioengineering systems can also have a significant effect on the habitat benefits. Various species of willow are the most commonly used woody plants because of their excellent rooting capabilities, good overhanging cover and shade, good nesting habitat for some species of birds, and some cover for mammals, other species offer better food sources for land animals. Soil bioengineering designs incorporate plants that provide the best habitat benefits for target species (Sotir 1997c).

Johnson Creek Relocation and Restoration

Johnson Creek is located in a highly urbanized area of Portland (Oregon), with land uses ranging from heavy industry to low-density residential. It is a third-order stream with a 100-year discharge at the project site of about 4,400 cfs. Flood control efforts during the 1930s enlarged, but did not straighten the stream. A survey of Johnson Creek revealed that with few exceptions, streambanks are stable, heavily vegetated, and provide excellent riparian habitat and overhanging cover for the stream.

The Oregon Department of Transportation (ODOT) proposed relocating a section of Johnson Creek in the Town of Milwaukie for bridge and highway construction (Figure 1). The relocated section would be about 20% shorter than the existing channel with a commensurate increase in gradient. The Johnson Creek Corridor Committee, created because of concerns over degraded water quality and aquatic habitat and with an interest in restoring an anadromous fishery, was worried about potential impacts of the stream relocation. The relocated stream reach is in a highly visible location, and the riprap channel proposed by ODOT would present a stark, sterile appearance and cause further loss of habitat and aesthetic value.

Robbin B. Sotir & Associates, Inc. (RBSA) was retained by ODOT to evaluate the proposed channel design for stability and for potential impacts to aquatic and riparian ecosystems and to modify the design as needed to address the concerns voiced by the Johnson Creek Corridor Committee. The review determined that the proposed trapezoidal channel cross-section shape and gradient were too uniform and that the floodplain berms were too high. RBSA

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Figure 1. Johnson Creek after realignment activities.

recommended changes to the channel to improve stability, water quality, and habitat value (Sotir and Nunnally 1995). The channel cross-section was altered by lowering floodplain berms, incorporating a sub-channel to convey bank overflows, and constructing a low-flow channel to concentrate flows during the summer months. A pool-riffle sequence was created by widening the sub-channel and raising the invert by one foot in cross-over reaches and lowering the invert by one foot in outside meander sections.

Streambanks were riprapped to the ordinary high-water elevation in the outside bends. Banks above were soil bioengineered, using vegetated geogrids. Siltation constructions using live materials were installed on the lowest floodplain berm adjacent to the sub-channel to provide cover for waterfowl and overhanging cover for fish. The upper bank was protected with brushmattress.

The soil bioengineering systems were installed in the winter of 1993 and spring of 1994. During the early spring and before the plants had established growth, the site experienced a 1,750 cfs flood with mean velocities of 6-7-feet-per-second and maximum velocities estimated in excess of 10-feet-per-second. The soil bioengineering systems were secure, and by the end of the growing season they were providing excellent bank protection and habitat benefits (Figure 2).

Buffalo Bayou Bank Stabilization and Aesthetic Improvement

Buffalo Bayou upstream of Sheperd Drive is the only stream of any size in Houston (Texas) that has not been channelized for flood controls. The watershed of Buffalo Bayou is almost totally urbanized, and Addicks Reservoir was constructed upstream to alleviate flooding. The combination of natural flooding and operation of the flood gates at Addicks results in abrupt rise and fall of the water level in the bayou coupled with prolonged periods of both high- and low-water levels. These hydrologic conditions, combined with sandy and silty soils with little cohesion, have resulted in widespread erosion and large streambank failures.

Several soil bioengineering streambank protection projects were built on Buffalo Bayou between 1990 and 1995 (Nunnally and Sotir 1995; Gray and Sotir 1996). The 1990 sites survived one of the largest floods of record in 1992 without damage. The installation described here was constructed in 1992-93 following that flood.

The project site, located in an outside bend, is 280 feet long and its height varies from 25-35 feet. Due to the receding bank, over 20 feet of land had been lost (Figure 3). The bank recession was caused by a combination of mass slope failure and streambank erosion. The instability of the steepened slope was aggravated by the presence of fine sands and seepage of 200-2,000 gallons per day from the bank face. While the main goal for this project was to stabilize the bank and stop the erosion, the client was also interested in the restoration of the riparian zone, aesthetic improvements, and the ability to maintain a view to the Bayou.

To achieve long-term bank stabilization, a foundation of wrapped concrete rubble was installed in a 7-foot deep toe trench. A fill slope with a grade of 0.5 H:1V was reconstructed above this foundation. The fill was constructed in 2-foot lifts wrapped with a geogrid. Thick layers of brush long enough to extend from the undisturbed soil at the back

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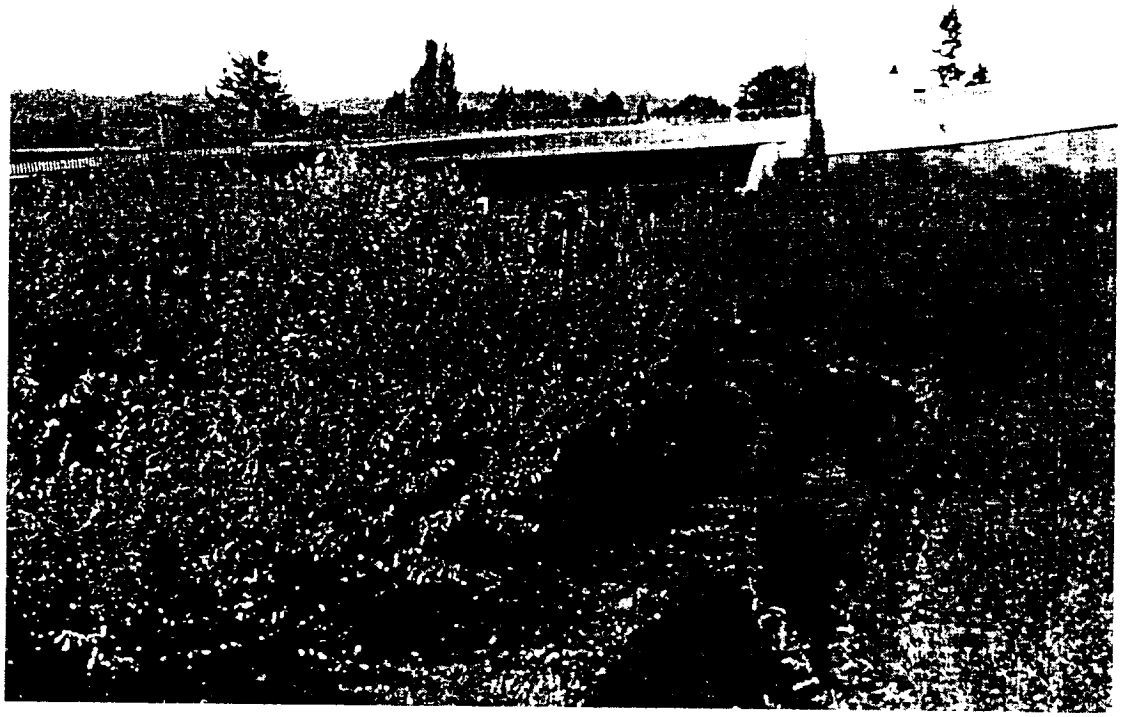

Figure 2. Johnson Creek four years after construction.

Figure 3. Buffalo Bayou erosional failure after a flood event.

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of the bench and protrude several feet beyond the slope face were placed between each wrapped soil layer. The overall constructed height was 42 feet with the upper half being at 0.25H:1V. Because continued seepage would have substantially reduced the safety factor, it was necessary to install vertical chimney drain construction to conduct the water into a gravel trench drain that discharged into the bayou. Since construction, the site has experienced several floods and has remained stable; meanwhile, the installation is developing into a dense riparian buffer of native and naturalized species (Figure 4).

Little Sugar Creek Stabilization, Habitat Restoration & Flood Control

This 4,650-foot section of Little Sugar Creek is in the Huntington Farms Park area in the City of Charlotte (North Carolina). This linear park, located along the creek in a predominately residential neighborhood, is owned by the City of Charlotte and maintained by Mecklenburg County Storm Water Services.

Like most other streams in Charlotte, Little Sugar Creek was channelized to improve drainage in the early 1900s, and it has been dredged and snagged several times since then, often leaving the channel without any vegetative cover. The stream drains much of eastern and central Charlotte, and the watershed is highly urbanized. The frequent flooding and high peak discharges caused significant bank erosion and channel enlargement. The immediate area had also been used as a constructed landfill in the past. At several locations, bank erosion had uncovered construction debris burial sites containing tree trunks, waste construction materials, and miscellaneous organics (Figure 5).

An interdisciplinary team with expertise in hydrology, surveying, geotechnical and aquatic science, fluvial geomorphology, and soil bioengineering was assembled to develop a restoration and stabilization project. Goals included: bank stability, erosion protection, aquatic habitat enhancement, water quality and aesthetic improvement, community education and economic savings.

Design and cost information studies were initiated in April 1996. Erosional bank failures along the creek were evaluated, typed and matched with appropriate solutions. From this, final plan and specification documents were prepared. Initially, riprap rock was reduced or completely eliminated along the toe. This dramatically reduced the project costs. Soil bioengineering methods such as live fascines and brushmattress were employed in different configurations along the banks. Construction was completed in March 1997. Four months after installation, Little Sugar Creek experienced a flood that exceeded the 100-year event. The project sustained no damage. While this project is a very new installation, it has become well-vegetated, offering enhanced riparian benefits, overhanging cover, aesthetic improvements and bank stability (Figure 6). The instream habitat structures (current deflectors and rocks) have also been performing well, producing a variety of scour hole cover and resting areas for fish.

Figure 4. Buffalo Bayou five years after construction.

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Figure 5. Little Sugar Creek erosion failures before construction.

Figure 6. Little Sugar Creek in the first growing season after construction.

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Long Leaf Hills/Hewletts Creek Stabilization, Aesthetic and Habitat Enhancement

This stretch of Long Leaf Creek is located in a residential neighborhood in Wilmington (North Carolina) known as Long Leaf Hills Subdivision. Increased stormwater runoff due to urbanization of the watershed and frequent flooding in the lower section have caused significant bank erosion and channel enlargement. Bank seepage and uncontrolled overbank runoff also contributed to bank failure (Figure 7 and 8). The creek has been used as a dump site for organic garden debris which kills the bank vegetation and has worsened erosion. Public meetings focused community concern on existing conditions and spurred interest in stabilization and restoration based on ways that residents wanted to use and enjoy the creek in the future.

Kimley-Horn & Associates, Inc., the prime consultant, and Robbin B. Sotir & Associates, Inc. prepared six conceptual alternatives which included a simple intermediate action for cleanup and stabilization, grass, riprap rock and concrete liners, box convert, and soil bioengineering. Alternatives were matched against 11 critical issues (Table 3). Soil bioengineering was selected by the neighborhood as it fulfilled all the criteria. The project is currently in the final design stages. Construction is scheduled to start in the fall of 1998 and is expected to be completed by late winter of 1999. Monitoring will be performed after construction to evaluate the stabilization and restoration development of Long Leaf Hills/Hewletts Creek (Sotir 1997a).

Summary

Urban water restoration and stabilization projects involve multiple objectives. In addition to controlling erosion in a cost-effective manner, we are increasingly concerned with water quality, habitat, aesthetics, recreational use and other environmental objectives. Soil bioengineering designs that employ woody vegetation meet these environmental objectives better than other types of streambank protection, especially when integrated with other technology. The successful retrofit applications of soil bioengineering on urban waters discussed in this paper indicate that this approach to stabilization and restoration is successful.

Figure 7. Hills/Hewletts Creek/Long Leaf failure conditions

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Figure 8. Long Leaf/Hills/Hewletts Creek failure conditions

Table 3. Long Leaf Hills/Hewletts Creek Alternatives and Critical Issues

Critical Issues	ALT. #1 Intermediate Action	ALT. #2 3:1 Side Slopes Grass Lining	ALT. #3 2:1 Side Slopes Riprap Rock	ALT. #4 2:1 Side Slopes Concrete Lining	ALT. #5 Reinforced Box Convert	ALT. #6 Soil Bioengineering
Stop Erosion & Stabilize Banks		•	•	•	n/a	•
Clean Out Trash & Debris	•	•	•	•	•	•
Remove Fallen Trees	•	•	•	•	•	•
Safer & Healthier Area		•				•
Control Flooding		•	•	•	•	•
Timely Project Completion	•	•	•	•	•	•
Environmental Improvement		•			•	•
Aesthetically Enhancing		•			n/a	•
Meets Bank Stability & Hydraulic Efficiency		•		•	n/a	•
Minimize Property Loss	•				•	•
Financial Feasibility	•	•	•	•	•	•

Adapted from Kimley-Horn & Associates (Sotir 1997a)

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Restoration of the Waukegan River Through Biotechnical Means

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Introduction

Many urban Illinois streams have been degraded as a result of streambank erosion, increased urban runoff, and increased channelization. Biotechnical stream stabilization techniques (BSST - Structures added to vegetation) were implemented on the Waukegan River to reduce the sediment load discharge to Lake Michigan originating from the river's eroded streambanks. The Waukegan River is located 30 miles northwest of Chicago, Illinois, in the City of Waukegan, Illinois (Figure 1). Best management practices (BMPs) were implemented on the Waukegan River in Washington Park and Powell Park, located in Waukegan. The Waukegan River Restoration Project was created to demonstrate whether the biotechnical techniques utilized were an effective means of resolving streambank erosion. The project was funded in part, by the United States Environmental Protection Agency (U.S. EPA), under the Section 319 Nonpoint Pollution Program of the Clean Water Act.

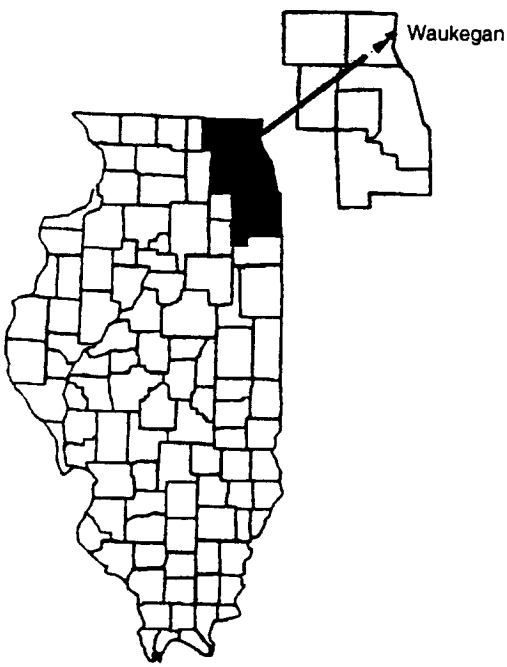

Figure 1. Waukegan River restoration project Section 319 National Monitoring Project.

At the selected severely eroded streambank sites, BSSTs were a more cost-effective and environmentally sensitive means of reducing nonpoint source (NPS) pollution than traditional approaches (i.e., rip rap, concrete lining).

Biotechnical Designs

Installation of the first BSST occurred on the North Branch of the Waukegan River in Powell Park and in Washington Park during the fall of 1991. Lunkers and A-Jack structures were installed in Powell Park, while lunkers with stone were installed in Washington Park (Figure 2). On the two lunkers installations, vegetation (willows, dogwoods, grasses, and other wetlands plants) were placed into the lower, middle, and upper zones of the lunkers structures. The structures utilized were chosen to enhance in-stream habitat and provide a structural base for riparian revegetation of the bank. Advantages and disadvantages of using lunkers with vegetation are listed in Table 1.

The next installations of BSSTs were on the South Branch of the Waukegan River, in the fall of 1994, to control severely eroded streambanks in Washington Park. To address these eroded streambanks, lunkers, stone, dogwoods, willows, and grasses were installed. Other BSSTs that included coir coconut fiber rolls, willows, and grasses were implemented to treat specific small streambank erosion sites on the South Branch.

In the winter of 1996, seven low stone weirs (LSWs) formed by granite boulders were installed to create a series of pool/riffle sequences to enhance in-stream habitat on the Waukegan River. These LSWs were constructed to help resolve a lack of water depth, limited cobble substrates, and limited stream aeration in order to enhance the aquatic community in the Waukegan River at Washington Park.

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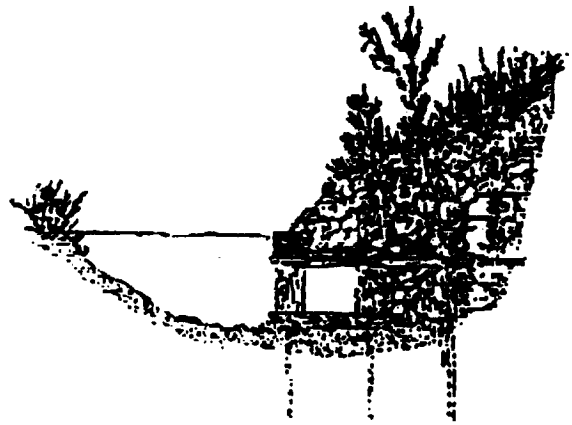
Lunker Installation Design



A-Jack Design and Installation

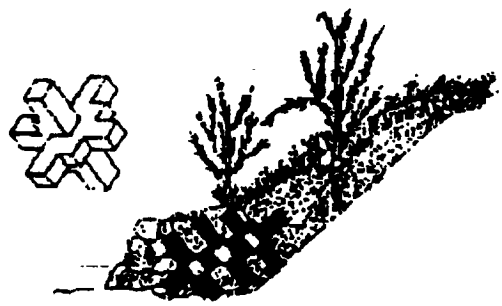


Figure 2. Biotechnical designs.

Table 1. Advantages and Disadvantages of Using Lunkers with Vegetation.

	Advantages	Disadvantages
A.	Provides greater public access to stream.	During first year, maintenance and revegetation are critical to project stability.
B.	Appearance of a natural stream functioning in an urban par is more appealing to the public.	Labor for lunker construction and installation is greater than riprap bank protection.
C.	Lower cost of installation.	
D.	Greater fishery benefits by increasing aquatic habitat for gamefish.	
E.	Maintenance operation requirements are revegetation, not construction activities.	

Monitoring

The Illinois EPA and the Illinois Department of Natural Resources are jointly monitoring the effectiveness of the biotechnical streambank techniques implemented on the Waukegan River. The U.S. EPA's National Monitoring Program (NMP) documents environmental benefits resulting from the BMPs implemented on the Waukegan River.

On the South Branch of the Waukegan River, protocols of the NMP were followed to detail the response of the stream fishery, the macroinvertebrate populations, and the in-stream physical habitat. The environmental quality of these three monitoring areas were evaluated utilizing the Index of Biological Integrity (IBI) for fisheries, the Macroinvertebrate Biotic Index (MBI) for benthic organisms, and the Potential Index of Biologic Integrity (PIBI) for in-stream habitat.

The monitoring plan divided the South Branch of the Waukegan River stream reach (Figure 3) into an upstream control (S2) and a downstream bank erosion site (S1) for biotechnical stabilization and in-stream habitat enhancement. This reach was chosen because no large ravine system transported urban runoff onto the stream between S1 and S2.

Between 1994 and 1997, the Waukegan River was monitored three times per year, once each in the spring, summer, and fall seasons. The monitoring activity documented

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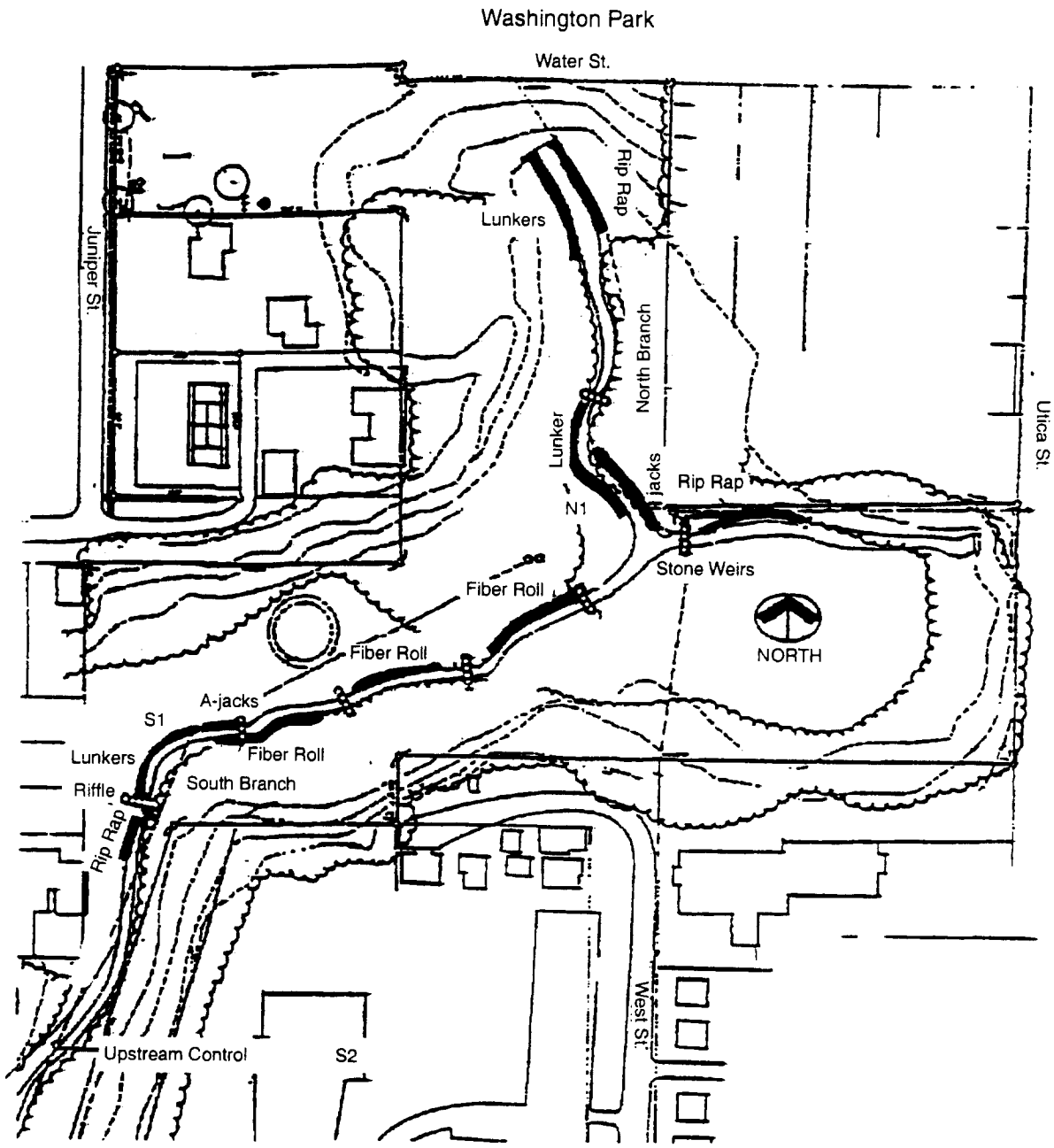

Figure 3. Map showing placement of erosion control techniques in the Waukegan River.

aquatic resources for one year before and one year after the biotechnical streambank stabilization and in-stream habitat enhancements were implemented. After the BSST application, the number of game fish species observed at S1 increased from four to five (Table 2). Following pool/riffle construction in 1996, the number of S1 game fish species increased to nine. Increased numbers of game fish and pollution intolerant fish species following the addition of the pool/riffle stream reach resulted in the IBI increasing from 26 to 35 (Table 3). The average number of fish sampled at S1 increased from 37 to 191 following lunker habitat enhancement and increased further, to 225, with the addition of the pool/riffle series. The upstream control (S2) remained a limited aquatic resource during the study period, with only 1-2 species present and an IBI of 28 or less during the entire monitoring period (Table 3). The average number of fish sampled at S2 varied between 16 and 69.

In 1996, the MBI indicated poor water quality at S2, with a value of 8.3 (Table 3), but better water quality at the S1 pool/riffle site, which remained in the non-limited classifi-

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Table 2. Comparison of the Fish Species and Abundance for S1 and S2, for 1994 to 1996

	1994		1995		1996	
	S1	S2	S1	S2	S1	S2
			Lunkers		Riffles	
Fish Species and Abundance						
Game Fish						
Coho					2	
Bluegill					9	
Largemouth bass	1				12	
Longnose dace					44	
Mottled sculpin			4		2	
Fathead minnow	4	2	64	4	16	
Creek chub	1		8		8	
Golden shiner	1	2	17		2	
White sucker			24	7	28	
Pollutant Intolerant Fish						
Black bullhead					3	
Green sunfish					8	
Mosquito fish	27	13	20	4	2	1
Goldfish	1				1	
Brook stickleback			1		1	
Ninespine stickleback	1				3	
Threespine stickleback	1		53	54	84	15
No. of species	8	3	8	4	16	2
Abundance of fish	37	17	191	69	225	16

Table 3. Comparison of the Mean Station Values of the Indices for S1 and S2, for 1994 to 1996

	1994		1995		1996	
	S1	S2	S1	S2	S1	S2
			Lunkers		Riffles	
IBI	25.82	22.18	25.33	26.00	34.67	28.00
MBI	6.64	7.26	6.26	6.31	6.99	8.26
PIBI	41.51	41.93	41.93	41.79	41.34	41.65

cation with a value of 7.0, even with the same stream waters as S2. The MBI indicates that water quality did not limit or degrade aquatic resources in 1994 or 1995 (Table 3).

Physical habitat evaluations found deeper pools at the S1 station, while the S2 site remained very shallow. The LSWs were designed to transport bedload and scour pools during high flow events. PIBI scores remained constant for all three years and for both the S1 and S2 sites, however, ranging between 41 and 42 (Table 3). The PIBI scores are predicated-on the absence of claypan or silt-mud substrates, the percentage of pools, and stream width. The S1 and S2 physical habitat had very little or no claypan substrate initially, which limited the expected change in PIBI.

A price comparison of the various types of bank stabilization are given in Table 4. The construction and installation techniques of lunkers make them relatively easy to use by volunteer citizens' groups. The relative costs of a rectangular concrete channel design, a riprap channel, a tri-lock channel, and lunker applications with vegetative stabilization can be estimated. The cost of a concrete culvert would include more design engineering support to determine possible offsite flooding effects. The design channel is 10 ft deep, 25 ft wide, and 300 ft long. The concrete channel would have a wall thickness of 10 inches.

This project demonstrated that BSSTs can be effective for reducing streambank erosion, by enhancing bank stability, and improving in-stream habitats. Incorporation of LSWs that created a pool/riffle series added to the in-stream physical diversity and resulting increased biodiversity. The project also demonstrated that LSWs are effective in increasing water aeration.

Streambank restoration is only one important step in improving the diversity of fish communities. LSWs provide additional pool depth and in-stream stone habitat necessary for higher quality fish communities in urban streams.

Table 4. Cost Per Foot of Various Applications

1. Lunker with vegetation	$27 per linear ft
2. Riprap with geofabric	$52 per linear ft
3. Tri-lock Channel	$165 per linear ft
4. Concrete Channel	$750 per linear ft

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Monitoring the Effectiveness of Urban Retrofit BMPs and Stream Restoration

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As a part of a larger Anacostia watershed restoration initiative, efforts have been underway since 1988 to restore Upper Sligo Creek. Over the past eight years, more than $2.0 million dollars have been spent on the restoration of Upper Sligo Creek and its environs. Upper Sligo Creek is a degraded, third order, urban Piedmont stream which flows through Montgomery County, Maryland. The general restoration strategy has featured the comprehensive employment of stormwater retrofits, instream habitat restoration, riparian reforestation, wetland construction and restoration, and native fish and amphibian reintroductions. Extended detention wet pond/marsh systems were employed on the basis of their ability to reduce pollutant loads and channel erosion and to create additional wildlife habitat. A prototype parallel pipe storm drain system was additionally used to divert first-flush stormflows away from an important feeder stream. Last, a wide variety of instream habitat enhancement structures such as rootwads, stone wing deflectors, boulder placement, log drop structures, etc. were employed. Restoration work was performed in Phase I and II of the three-phase project which covered the 1990-95 period. Biomonitoring of fish and macroinvertebrates was conducted before, during and after each construction phase. Physical habitat, hydrological and chemical conditions were monitored in Phase III. The number of established fish species residing in Upper Sligo Creek has risen from three species in 1988 to approximately 12 in 1997. Monitoring results were used to determine general retrofit effectiveness, adjust fish stocking strategies, document recruitment success and critique the overall effort.

Introduction

Attempts to restore the once highly degraded Upper Sligo Creek stream system exemplify the basic subwatershed restoration approach being employed throughout much of the urban, 400 kM^2 Anacostia River watershed. The completion of the Wheaton Branch stormwater management (SWM) retrofit facility in June, 1990 marked the beginning of a three-phased restoration project. The major objective of Phase I (1990-91) was to restore Wheaton Branch, Upper Sligo Creek's largest and most severely degraded tributary. The centerpiece of this effort was the three-celled, wet extended detention Wheaton Branch SWM pond/marsh. This SWM retrofit was designed to provide both a high level of water quality control and downstream channel erosion protection for a 326 ha (805 ac), 55% impervious catchment. Other major components completed under Phase I included restoration of 300 m of downstream aquatic habitat, the creation of two vernal pools for amphibian breeding habitat, and riparian restoration along a 350 m stream corridor.

Phase II (1992-94) restoration featured the completion of the University Boulevard SWM retrofit (a companion, two-celled wet extended detention pond/marsh). The SWM facility provides similar water quality and quantity control for a 162 ha (400 ac), 30% impervious drainage area. In addition, Phase II included: selective physical aquatic habitat restoration of approximately 7 km of the Upper Sligo Creek mainstem, construction of a 300 m-long parallel pipe stormflow diversion system along Flora Lane tributary, creation of a 0.1 ha marsh, riparian reforestation of 2 ha along Sligo Creek and the systematic reintroduction of 17 native fish species into Wheaton Branch, Flora Lane tributary and the Sligo Creek mainstem. Physical aquatic habitat conditions at 19 sub-project sites were enhanced via the employment of stone wing deflectors, boulder fields, rootwads, placed rip-rap, log drops, streambank bioengineering and cedar-tree brush bundles.

Because of the general lack of adequate surface stormwater runoff storage sites, physical aquatic habitat restoration of the Flora/Lane tributary necessitated a flow diversion approach. The prototype flow-splitting system was designed to divert stormflow generated from up to 90% of all one-hour storm events. Peak one-hour discharge from the 87 ha (216 ac), 50% impervious catchment is approximately 1.6 m3/s or 55 cfs.

Phase III (1994-95) included biological, physical habitat, hydrological and stream and pond water chemistry

[1] Project manager and co-investigator with James D. Cummins, Interstate Commission on the Potomac River Basin and James B. Stribling, Tetra-Tech, Inc.

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evaluations. No major restoration construction work was performed in Phase III.

Study Design

Between March, 1990 and June, 1995, macroinvertebrate and fish monitoring was performed at a total of 10 sites to help assess the success of stormwater retrofit and stream restoration work on Sligo Creek's aquatic communities. Over this six-year period, the number of sampling stations grew from four in Phase I to eight in Phase II and finally, 10 in Phase III. Of the 10 sites, four were located in the Sligo Creek mainstem, two in the Flora Lane tributary, two within the restored portion of Wheaton Branch, one in the unrestored Woodside Park tributary and one in the SWM control comparison stream (i.e., Crabbs Branch, located in the neighboring Rock Creek watershed). In addition, a similar headwaters area of the neighboring, semi-rural Northwest Branch served as the Piedmont reference stream. Upper Sligo Creek restoration areas and monitoring station network are shown in Figure 1.

Stream water quality grab sampling was conducted between May, 1994 and July, 1995 at the following sites: Wheaton Branch - W131; Sligo Creek - SL2, SL3, SL4; Flora Lane tributary FL1 and FL2; and Crabbs Branch - C131. Paired baseflow and stormflow water samples for laboratory analysis were collected at WB1 and FL1 between June, 1994 and July, 1995. Monthly pond water column sampling of Wheaton Branch Pond No. 3 and the Crabbs Branch SWM facility was performed between May and November 1994.[2] As part of Phase III, sediment grab sampling was conducted at six locations: Wheaton Branch Pond No. 3, WB1, SL2, Sligo Creek mainstern above Flora Lane tributary, SL4 and FL1. Stream thermal regime characterization via continuous temperature monitoring was performed between May and November, 1994 at the following locations: SL2, W131, FL2, SL4 and CB1.

Methods

Macroinvertebrate sampling of riffle and pool habitats was performed using a square foot Surber sampler and long-handled D-frame net (595 micron mesh opening). Three Surber samples and a single D-frame sample were taken from riffle and pool areas, respectively. Specimens were identified to the lowest practical taxonomic level. Five metrics were calculated in the study: taxa richness, Hilsenhoff Biotic Index, EPT, percent contribution of dominant taxon and shredders/total.

Fish sampling was conducted via backpack electrofishing. Sampling techniques followed procedures present in Plafkin, et al. (1989) and as described in Cummins (1989) and (1991). The Zippin (1956) three-pass depletion method was used for fish population estimation. In addition, one-pass electrofishing was performed to further evaluate fish dispersion, taxa richness and recruitment success in Upper Sligo Creek.

Spot baseflow and stormflow water quality readings were made in the field using a Horiba U-10, multiprobe water quality meter and a Hach TDS meter. Paired baseflow and stormflow samples were collected for WSSC laboratory analysis from Wheaton Branch and the Flora Lane tributary. Baseflow samples were collected by immersing a 20-L polyethylene carboy in an undisturbed pool area. Stormflow samples were collected using a modified suspended sediment sampler.

Pond water column samples were collected at established representative surface, mid-level and bottom depths using a 2.0-L Van Dorn sampler. At the Wheaton Branch pond, one 4-L water sample was collected for laboratory analysis at each of the following depths: 0.15, 0.61 and 1.22 m.

An EPA priority pollutant scan of stream (pool) and pond sediments was performed by first taking 8-L of fine sediment with a coring device. Samples from three-to-five locations at each site were composited and delivered to Gascoyne Laboratories, Inc. for analysis.

Continuous stream temperature monitoring was accomplished through the systematic employment of Ryan TempMentor recording thermograph thermometers.

Physical aquatic habitat conditions were visually evaluated using both methods described in Barbour and Stribling (1991) as well as the Rapid Stream Assessment Technique (Galli, 1996).

Results

Stormwater Pond Influence

Both the Wheaton Branch and Crabbs Branch SWM facilities exerted a strong influence on downstream hydrology, water chemistry, temperature, substrate particle size and stream bioenergetics. As expected, water quality in both ponds was typically highest at or near the surface and declined with increasing depth (Table 1).

During the Phase III study period, Wheaton Branch's 1.1 m release depth resulted in the periodic discharge of poorly oxygenated water high in organic materials and fine sediments (note: the pond's outlet structure was slightly modified in 1996 resulting in a mid-level release). Previous findings (Environmental Dynametrics, Inc., 1993) strongly suggested that during stormflow conditions this subsurface release functions as a siphoning device, effectively reducing the pond's overall pollutant removal efficiency. Of the stream sites monitored in Phase III, dissolved oxygen (DO) levels in Wheaton Branch were typically lower. DO concentrations there were below 5.0 mg/L on four out of the 30 sampling dates. The study's low stream DO reading

[2]Pond characteristics - Wheaton Branch: D.A. = 326 ha; imperviousness = 55%; permanent pool surface area 2.4 ha; bottom release design; maximum depth 1.75 m; constructed 1990; SAV absent; 24-36 hr ED control. Crabbs Branch: D.A. = 238 ha; imperviousness = 60%; permanent pool surface area = 3.1 ha; surface release design; maximum depth 2.60 m; constructed 1983. SAV (Hydrilla) covers approximately 75% of pond bottom; no formal ED.

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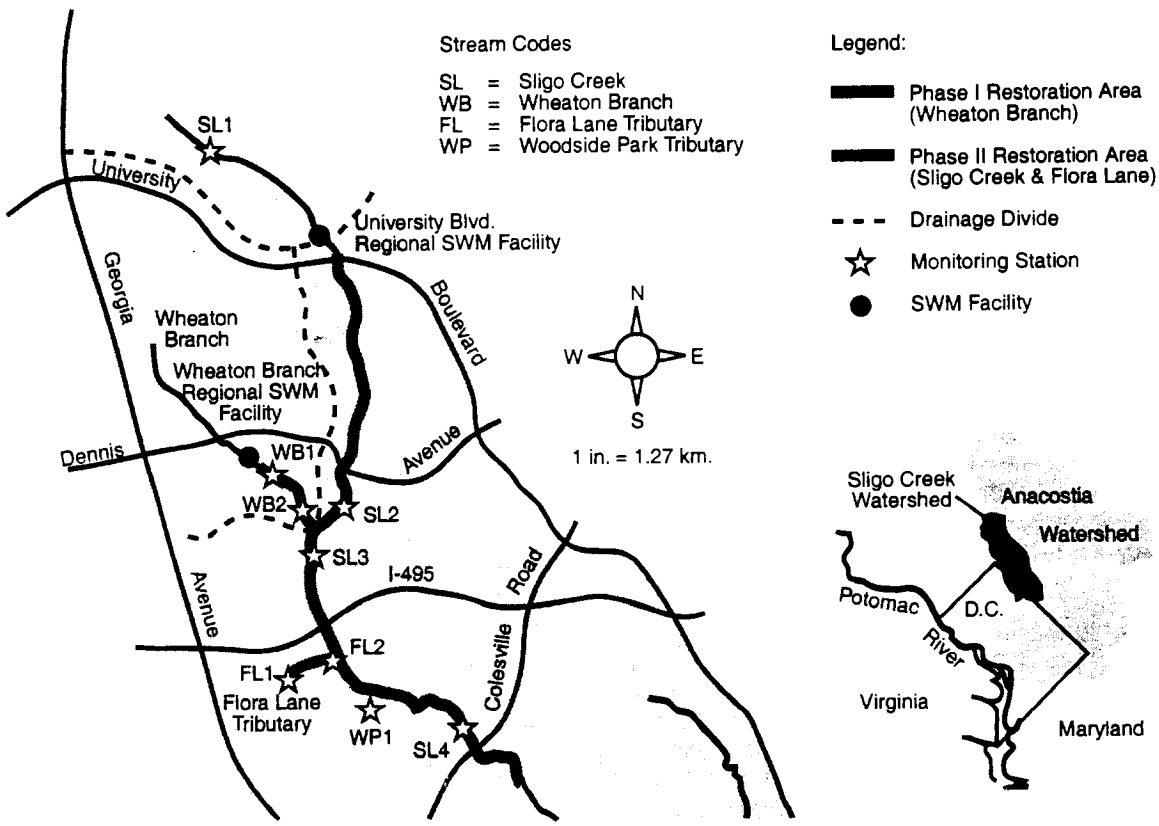

Figure 1. Upper Sligo Creek restoration and monitoring station network.

Table 1. Wheaton Branch and Crabbs Branch Pond Water Quality: June, 1994

Location & Date	Depth (m/ft)	Air Temp. (°C)	Water Temp. (°C)	DO (mg/L)	Field pH	Cond. (umhos/cm)	Turb. (NTU)	Secchi Depth (m)
Wheaton	0.15/0.5	32.0	28.5	12.13	7.30	146	16	0.50
Br. Pond	0.30/1.0		28.6	12.46	7.21	146	14	
No. 3	0.61/2.0		28.2	11.79	7.12	146	18	
(6/22/94)	0.91/3.0		27.5	5.93	6.72	147	17	
	1.22/4.0		26.5	1.20	6.53	210	34	
	1.45/4.5		26.0	0.23	6.21	227	34	
Crabbs Br.	0.15/0.5	32.0	28.4	12.73	8.77	300	11	0.72
SWM Pond	0.30/1.0		28.3	12.12	8.71	299	14	
(6/23/94)	0.91/3.0		27.8	10.44	8.51	294	13	
	1.22/4.0		27.0	3.75	7.26	298	14	
	1.83/6.0		23.0	0.22	6.60	423	139	
	2.59/8.5		25.2	0.11	6.57	595	87	

(2.87 mg/L) was recorded at both sites WB1 and W132 in June, 1994.

By comparison, Crabbs Branch's surface release design resulted in the discharge of warmer, yet clearer and more highly oxygenated water. The larger permanent pool surface area and volume and presence of extensive stands of Hydrilla verticillata (which cover approximately 75% of the pond bottom) contributed to Crabbs Branch's apparently better water quality performance.

Wheaton Branch and Flora Lane Tributary Stormflow Chemistry

Compared to Wheaton Branch, stormflow total suspended solids (TSS), total organic carbon (TOC) and biochemical oxygen demand (BOD) were generally slightly higher in Flora Lane. Median stormflow TSS, TOC and BOD concentrations were as follows: Wheaton Branch - TSS (20 mg/L), TOC (8 mg/L), BOD (9 mg/L); Flora Lane tributary - TSS (50 mg/L), TOC (10 mg/L), BOD(10mg/L). The

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median nitrate (NO_3) concentration was three times higher in Flora Lane (1.6 mg/L) than in Wheaton Branch (0.5 mg/L). Stormflow copper concentration ranges were nearly identical for both streams. The median stormflow copper concentration for both Wheaton Branch and Flora Lane was 20.0 ug/L. This median level was double that recorded under baseflow conditions. Mean stormflow total hardness concentrations for Wheaton Branch (80.3 mg/L $CaCO_3$) and Flora Lane (105.2 mg/L $CaCO_3$) were also considerably lower than under baseflow conditions.

Stream Sediment Chemistry

Results of the EPA priority pollutant scan revealed no high or unusual concentrations of pollutants in the sampled stream sediments and were deemed to not pose serious environmental toxic risks. Not surprisingly, the majority of contaminants found were associated with road runoff. For all metals detected, higher concentrations occurred in the Sligo Creek mainstern below the Flora Lane tributary confluence than above. For example, lead concentrations increased from 23 mg/kg above the Flora Lane confluence, to 50 mg/kg below. This enrichment is likely associated with the large volume of highway traffic and runoff from Interstate 495 and Georgia Avenue (MD Rte 97), which are conveyed via the Flora Lane tributary to Sligo Creek.

1994 Thermal Regime Characterization

Based on continuous water temperature monitoring results, the thermal regimes of the streams were generally categorized, per Galli (1990), as follows: 1) Sligo Creek mainstem - coolwater; 2) Crabbs Branch - coolwater bordering on warm; and 3) Flora Lane tributary - coolwater bordering on cold. Summer stream temperatures in all but the Flora Lane tributary regularly exceeded temperature levels considered optimal (i.e., less than 17-20° C) for many stonefly, mayfly and caddisfly species (Gaufin and Nebeker, 1973; Ward and Stanford, 1979; Fraley, 1979). Compared to Wheaton Branch, Crabbs Branch was typically 3-4° C warmer. This condition remained operative throughout the temperature monitoring period.

Physical Aquatic Habitat

Major aquatic habitat improvement occurred in Wheaton Branch following restoration work in April, 1991. Prior to this date, aquatic habitat at sites W131 and W132 was 49-56% of reference stream conditions. Following restoration, aquatic habitat at these two sites increased to 104-108% of reference. Similar improvements were documented in both the Flora Lane tributary and Sligo Creek mainstern upon completion of habitat enhancement work in February, 1994. Marked reductions in embeddedness levels were recorded throughout. Pre- and post- restoration embeddedness levels in Flora Lane fell from approximately 85% to 40%.

Macroinvertebrates

From Phase I to Phase II, both the number of individuals and number of taxa in Wheaton Branch and the Sligo Creek mainstern downstream of Wheaton Branch increased by approximately 50%. No discernible change was observed in the Flora Lane tributary. For the restored stream sites, the metric percent contribution of dominant taxon ranged from approximately 67-93% in 1990 spring samples to approximately 26-78% in 1995 (Table 2).

Fish

Between Phase I and III the number of established fish species increased as follows: Wheaton Branch - three to six; Flora Lane tributary - three to six; Sligo Creek mainstern - three to nine. Follow up, one-pass electrofishing results in 1996 and 1997 revealed that approximately 12 species are now established in the Sligo Creek mainstem. By comparison, Crabbs Branch and the reference stream support, 12-15 and 16-17 species, respectively. As seen in Figure 2, Index of Biotic Integrity (IBI) scores for restored sites SI-2, W131, WB2, SI-3, FL1, FI-2 and SI-4 all increased between Phase I and III (i.e., generally from poor to poor/fair). During Phase III, Crabbs Branch fish IBIs were consistently in the fair/good category.

Discussion

Monitoring results confirmed that the Upper Sligo Creek restoration produced several improvements in both biological and aquatic habitat conditions. These generally included: increases in the number of macroinvertebrate individuals (hence, improved food base for resident fish); reductions in percent contribution of dominant taxon; an increase in the number of established fish species from

Table 2. Calculated Macroinvertebrate Metric Values: Spring 1995* (modified from Cummins, et al., 1997)

Monitoring Site	Taxa Richness	Hilsenhoff Biotic Index	EPT	Percent Dominant Taxa	Shredders (Total)
SL1	7	7.8	2	63	0
SL2	10	6.5	3	47	0.01
SL3	9	7.0	3	26	0
SL4	10	6.9	3	30	0.12
WB1	11	6.8	3	46	0.006
WB2	7	6.3	3	78	0
FL1	2	8.0	0	67	0
FL2	6	7.3	2	48	0.05
WP1	3	8.1	0	61	0
CB1	10	7.0	3	43	0.007

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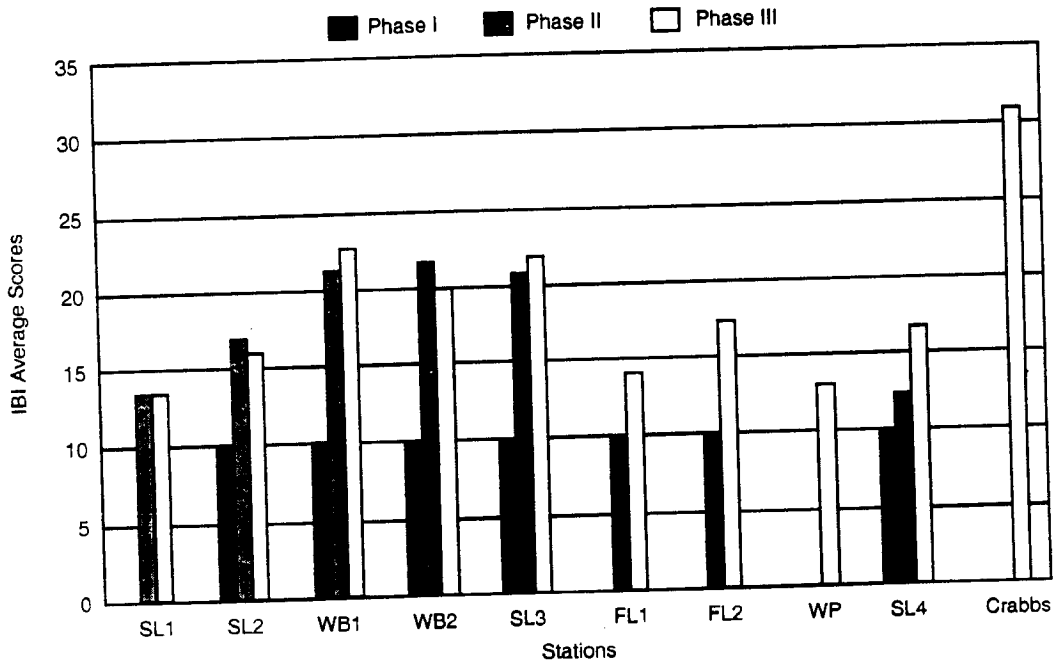

Interpretation: 10-21 = Poor, 22-33 = Fair, 34-45 = Good, 46-50 = Excellent

Modified from Cummins, et al., 1997.

Figure 2. Summary: Upper Sligo Creek Phase I-III average fish IBI scores (modified from Cummins, et al., 1997).

three in 1988 to approximately 12 in 1997; reductions in streambed embeddedness levels and increases in overall riffle and pool quality; enhanced streambank stability; dramatic reductions in the amount of sediment, trash and debris present in the streams; a reduction in the incidence of fish deformities, skin erosions, lesions and tumors (DELTs) from 11% in Phase I to less than 3% in Phase III, and the ability to support relatively pollution intolerant fish species such as the mottled sculpin (*Cottus bairdi*), rosyside dace (*Clinostomus funduloides*) and northern hogsucker (*Hypentelium nigricans*) for periods as long as three years.

Despite these major improvements, the gain in Upper Sligo Creek's aquatic ecological health was generally limited to a shift from a very poor stream system to a fair one. At the end of Phase III, conditions were well below those found in the reference Piedmont stream. They were also generally lower than those present in Crabbs Branch. In addition, it was evident that both water quality and high runoff volumes remain a problem, particularly in the Wheaton Branch and Flora Lane tributaries. With regard to Wheaton Branch, the Wheaton Branch SWM facility's water quality control limitations appear to be hindering the re-establishment of a more diverse fish and macroinvertebrate community. In Flora Lane, the parallel pipe system is undersized. Consequently, approximately one to three times per year, high runoff discharges produce major streambed scouring. Episodic discharges of petroleum products into the stream, as well as the failure to perform routine sediment and debris removal at the parallel pipe system's control weir, have further limited stream recovery.

Conclusion

The effort required to shift the level of a severely degraded, intensely developed urban stream system to a higher level is not well understood, nor are results always linear. In the case of Upper Sligo Creek, the post-restoration aquatic community is still undergoing changes. Thus, additional monitoring, perhaps for another two to three years, may be required to more fully explain the recovery. Finally, continued efforts to reduce water quality and quantity-related problems are critical for further biological improvements and the ultimate restoration of the fish community to near-reference conditions.

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Urban Water Quality Monitoring and Assessment Approaches in Wisconsin

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The Wisconsin Department of Natural Resources is implementing a long-term urban monitoring strategy designed to support Wisconsin's Nonpoint Source and Stormwater Permit Programs. The purpose of the monitoring is to help ensure that management programs will improve the quality of Wisconsin's urban streams in the most cost-effective manner. At the core of the strategy are seven questions that we try to answer as part of preparing every urban nonpoint source control plan. All of the plans are enhanced with site-specific monitoring data for the first three questions, but the high cost of collecting monitoring data for the remaining four questions limits the data collection to special urban monitoring projects. Results are available for 26 special monitoring projects. To improve our answers to all of the questions, we are planning about 28 new special monitoring projects for the next five years. An urban runoff model and stormwater management manuals are used to transfer the results of special projects to other urban watersheds.

One or more types of monitoring data are being collected to answer each question. Biological data is collected in every urban watershed to answer the first two questions and part of question 3. The first three questions are: 1) what are the designated uses of the streams, 2) what are the problems in the stream, and 3) what are the pollutants and/or habitat factors degrading the streams? Special monitoring projects using biological, chemical and physical data attempt to answer the last four questions and part of question 3. The part of question 3 answered by special monitoring is what, if any, potentially toxic pollutants are degrading the streams. The last four questions are: 4) what are the sources of the pollutants, 5) what are the goals for reducing the pollutants and changing other factors degrading the stream, 6) what management alternatives will achieve the goals, and 7) what did the implementation of practices improve in the streams?

Introduction

Urban runoff has degraded many of Wisconsin's streams (Masterson and Bannerman, 1994; Simonson and Lyons, 1993). To improve and protect the quality of urban streams and other resources, Wisconsin's State Legislature created a Nonpoint Source Program in 1978. The program is implemented through "priority watershed projects," which include the preparation of a priority watershed plan. The priority watershed plan assesses nonpoint and other sources of water pollution and identifies best management practices needed to achieve the designated uses of the water resources.

The watershed plan guides the implementation of the best management practices. The Wisconsin Department of Natural Resources (WDNR) and the Department of Agriculture, Trade, and Consumer Protection administer the program, while the local units of government implement the plan. State funds are provided to cost-share the implementation of the best management practices recommended in the plan. Approximately 10 million is made available to municipalities every two years. Approximately 6 million is for the installation of urban practices and the remaining 4 million dollars is used for such activities as designing the practices, stormwater management plans, developing stormwater ordinances, and developing utility districts.

A priority watershed plan could not be prepared without the results from some type of urban monitoring activity. Results of the monitoring help us make the best management decisions to ensure that designated uses of the streams are achieved for the least cost possible. Monitoring data is used to strengthen our confidence in such decisions as identification of the pollutant sources and the selection of management alternatives. In response to this need for data, the WDNR has developed an urban monitoring strategy supported by two types of monitoring activities.

The purpose of this paper is to describe the monitoring strategy and some of the results from the monitoring activities. Information from the Lincoln Creek Subwatershed part of the Milwaukee South Priority Watershed Plan is described as an example of how the monitoring data is used to prepare the chapters in a plan. Future monitoring activities proposed for the strategy are also discussed.

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Seven Stormwater Management Questions

The urban monitoring strategy is based on answering seven stormwater management questions (Table 1). Each question is related to the information needed for a chapter in the watershed plans. Chapter IV in the priority watershed plan for the Milwaukee River South Priority Watershed Project is entitled "Water Resources Conditions, Nonpoint Sources and Water Resource Objectives" (WDNR,1991). Designated uses and water resource objectives are usually the same. Results from questions 1 to 5 are needed to prepare this chapter. Answers to question 6 are helpful for making the management recommendations in Chapter V, which is entitled "Nonpoint Source Control Needs." The last question matches with the last chapter, Chapter VIII, entitled "Water Quality Evaluation Monitoring." Titles and order of the chapters might vary between plans, but they all cover the same types of information. These same questions would probably apply to almost any water resource management effort.

Table 1. Seven Stormwater Management Questions Used To Design Urban Monitoring Activities.

Question No.	Questions
1.	What are the designated uses of the urban streams?
2.	What are the problems in the stream?
3.	What are the pollutants and/or habitat factors degrading the urban streams?
4.	What are the sources of the pollutants?
5.	What are the goals for reducing pollutant loads and changing other factors?
6.	What management alternatives will achieve goals?
7.	What did the implementation of the practices improve in the urban streams?

All of the questions are important. The quality of the answer to a question depends to some degree on how good the answer is to the previous question. The order of the questions is the order in which they are usually answered. For example, selecting the best management practices without identifying the sources of the pollutants increases the risk of wasting money on the wrong practices.

Answers to the seven questions are also helpful in the implementation of the U.S. Environmental Protection Agency's Stormwater Permit Program. Priority watershed plans are available for many metropolitan areas in Wisconsin. Most municipalities required to have a stormwater permit are in a priority watershed. So far, some of the permit requirements overlap with the management actions specified in the watershed plans.

Two Types of Monitoring Activities Supporting the Urban Monitoring Strategy

Two types of monitoring activities are essential parts of the urban monitoring strategy. Results from the two types of monitoring activities are used to answer the seven management questions for all of the priority watershed projects. One type of monitoring is done for every priority watershed project. Biological data is collected in every stream to answer questions 1, 2, and parts of 3. Information about the designated uses, the problems in the urban stream, and the reasons for any problems are very site-specific. Parts of question 3 assessing the problems caused by conventional pollutants and a degraded fish habitat are included in the monitoring done for every stream.

There is no substitute for collecting good biological data in every watershed project. In most cases, it cost about $500 to collect the biological data in each urban stream. One or two sites in every stream are selected for fish, fish habitat, and macroinvertebrate sampling. The biological sampling for questions 2 and 3 is always done during the planning phase of a project, while the answer to question 1 is sometimes determined before the beginning of a watershed project.

The other type of monitoring provides data to answer questions 4, 5, 6, 7, and the part of question 3 evaluating the role of potentially toxic pollutants. These "special urban monitoring projects" collect data at a few selected sites that are then extrapolated to other urban areas. Special urban monitoring projects provide the kind of data needed by every priority watershed project, but which would be too expensive and time-consuming to collect for every project. Concentrations of zinc measured in street runoff in Madison, for example, is used to estimate zinc loadings from streets in Milwaukee. Testing the effectiveness of a best management practice in every priority watershed project would not only be unnecessary, but would cost over $100,000 to properly test each device. Results from three special monitoring projects were used to help answer the stormwater management questions for the Lincoln Creek Subwatershed.

Answers to Seven Questions for Lincoln Creek Subwatershed

Answers to the seven questions for the Lincoln Creek Subwatershed provide an example of how the information is presented in many of the priority watershed plans. Special urban monitoring project data used for the Lincoln Creek Subwatershed is also typical of the stormwater data available to a number of the priority watershed plans prepared before 1993.

Lincoln Creek Subwatershed is the largest urban subwatershed in the Milwaukee River South Watershed, draining 12,600 acres (18.8 sq. miles). Information about Lincoln Creek was collected as part of preparing the Milwaukee River South Priority Watershed Plan (WDNR, 1991). Residential land uses dominate this totally urban subwatershed. High density residential areas occupy 35% of the subwatershed, while 12% of the subwatershed is industrial. Lincoln Creek is almost entirely channelized with about one-third of the channel being concrete lined.

Lincoln Creek Answers - Questions 1 to 3

As for all the urban streams in the Milwaukee South Priority Watershed Project, biological sampling was done in

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Lincoln Creek to answer questions 1, 2 , and parts of question 3 (Table 2). Electro-shocking of fish was done to develop an assessment of the fish community in Lincoln Creek. The WDNR's stream system habitat rating form (Ball 1982) was used to characterize the habitat in the stream. Thirteen factors, such as low flow, depth of pools, bank vegetation protection, lower bank deposition, and characteristics of bottom substrate and cover are ranked to determine the quality of the habitat. Habitat data are very important because they are a large factor in determining the potential fish species, composition, abundance, and age structure. Results from dissolved oxygen, temperature, pH, and bacteria surveys were combined with the ranking from the Hilsenhoff Biotic Index (HBI) for macroinvertebrates (Hilsenhoff 1982) to assess the water quality of the stream. All of the environmental data is then used to classify the stream.

Procedures developed by the WDNR are used to classify the urban streams for fish and other aquatic life (Ball, 1982; WDNR, 1995). Although the procedures are designed to provide a legal classification of a stream, classifications prepared for the priority watershed projects do not follow all of the required steps and, therefore, carry no legal authority. The stream use classes are (a) cold water communities, (b) warm water sport fish communities, (c) warm water forage fish communities, (d) limited forage fish communities, and (e) limited aquatic life. Recreational stream use classifications are also defined. For the purpose of designating fish and aquatic life uses, the biologist must decide if the factors limiting the ability of a stream to support certain uses are controllable or uncontrollable. If a controllable factor, such as urban runoff, is limiting the uses of a stream, the biologist can assume the urban runoff will be controlled to some degree when deciding what the potential uses of the stream should be. Although the procedures provide more objectivity to the process of classifying streams, professional judgement usually enters into the final use class selection.

Table 2. Answers to the Seven Stormwater Management Questions for Lincoln Creek Sub-watershed (from WDNR, 1991)

Question	Answer to Questions
1. What are the designated uses?	*Fishery Use*: Below Teutonic Ave. - warm water sport fish; other natural reaches - limited forage fish; and concrete lined reaches - limited aquatic life *Recreational Use*: All reaches - partial body contact
2. What are the problems in the stream?	*Fishery Use:* Species diversity - 2 (Ref. stream -20) *Macroinvertebrates:* Severely impaired *Recreational Use:* Partially meeting use
3. What are pollutants and/or factors degrading the stream?	*Pollutants*: Sediment, potentially toxic pollutants (eg. lead, zinc, and copper) in water column and bottom sediments, *bacteria, and low dissolved oxygen.* *Factors*: Poor habitat, flashy flows, and concrete lining.
4. What are the sources of pollutants?	*Sediment*: Established urban area - 29%; construction sites - 64%; and streambanks - 7%. *Lead:* High density residential - 33%; industrial - 32%; Multi-family residential - 19%; and commercial - 14% *Bateria and low dissolved oxygen* - no entry
5. What are goals for reducing pollutant loads and changing other factors?	*Flow Rate and Volume*: Reduce enough to control bank erosion and scour. *Sediment:* A 50% reduction in sediment *Lead:* 40% to meet acute toxicity standards at outfalls and 50% to meet chronic toxicity standards in stream.
6. What management alternatives will achieve the goals?	*Sediment:* Construction site erosion controls designed to reduce the sediment by 75% *Lead:* Wet detention ponds or their equivalent to control all of the runoff from critical land uses (industrial, commercial, freeways, high density residential, and multi-family residential). *Flow Rate and Volume:* No entry
7. What did the implementation of the practices improve in the stream?	Develop long-term biological, chemical, and physical monitor ing program.

Not all of question 3 was answered by biological monitoring. Although problems caused by excessive sediment, and sometimes high nutrient loadings, can be identified by the fish habitat surveys, some grab samples of Lincoln Creek water were used to identify the presence of potentially toxic pollutants. We recommend raising public awareness about the potential problems with toxic pollutants and bacteria by collecting grab samples below a storm sewer outfall during three different runoff events. These samples should be analyzed for as many of the pollutants found in stormwater as possible, such as heavy metals and fecal coliform bacteria.

Lincoln Creek Answers - Questions 4 to 6

Lead is the potentially toxic pollutant that is assumed to represent all of the other potentially toxic pollutants in the answers to questions 4 through 6. Lead is also important to part of the answer for question 3. Extensive stormwater monitoring of eight storm sewer outfalls in the City of Milwaukee identified the types of potentially toxic pollutants that might be in the stormwater discharging to Lincoln Creek

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(Bannerman, 1983). Results of this special urban monitoring project were available for samples collected at high density residential, medium density residential, commercial strip, and shopping center monitoring sites. Total recoverable lead event mean concentrations exceeded the acute toxicity standard (hardness of 100mg/l) of 170 ug/l for 90% of the runoff events sampled at a commercial landuse site (WDNR, 1989). Although the WDNR does not currently regulate stormwater discharges using numeric effluent limitations, acute and chronic toxicity standards applied to point source discharges for industries and municipalities are useful to characterize the potential importance of different pollutants to the quality of urban streams. Bottom sediment samples collected for a special urban monitoring project in the nearby Menomonee River indicated that all urban stream bottom sediments are probably contaminated with heavy metals (Dong, 1979).

Sources of lead and sediment for the established urban areas were estimated using an urban runoff model called *Source Loading and Management Model* (SLAMM) (Pitt, 1989). SLAMM is widely used in Wisconsin as a planning tool to better understand sources of stormwater pollutants and their control. Percent contributions listed in the answer to question 4 are a lot more credible because SLAMM was first calibrated with the data from the 1983 Milwaukee stormwater monitoring project. Once the model was calibrated it was also used to estimate the pollutant reduction goals. An average annual lead concentration calculated with SLAMM for all the outfalls in Lincoln Creek subwatershed was compared to the acute toxicity criteria for lead. About a 40% reduction in lead loading was needed from all the critical landuses in the subwatershed to meet the acute toxicity standard in the stormwater discharged from the outfalls. Concentrations measured in the Milwaukee River were used to determine the exceedances of the chronic criteria in the stream.

Two years of samples collected at the inlet and outlet of a wet detention pond in Madison, was the basis of the answer developed for question 6 (House, 1993). The results of this special urban monitoring project confirmed that about a 90% reduction in sediment and about a 60% reduction in lead could be achieved with wet detention ponds.

Lincoln Creek - Question 7

Lincoln Creek is part of an intense evaluation monitoring effort in Wisconsin. Comprehensive biological, chemical, and physical monitoring is being done for at least a ten-year period. Results from Lincoln Creek will be used to evaluate the benefits of implementing best management practices in other urban streams. All of the pre-practice installation monitoring has been completed for Lincoln Creek.

Special Urban Monitoring Projects

Results from the three special urban monitoring projects referenced above were available in time to enhance the answers to the seven stormwater management questions in the Lincoln Creek Subwatershed. None of these projects were conducted as part of preparing the Milwaukee South Priority Watershed Plan. Determination of pollutant reduction goals would have been more difficult if the monitoring data had not been available from the 1983 Milwaukee stormwater monitoring project. Although there was insufficient data in these three projects to completely defend the answers to the questions, it was enough to begin an implementation effort for Lincoln Creek.

A total of 26 special urban monitoring projects have been conducted by the WDNR over the last 17 years. Each one of the projects was selected to help answer one or more of the seven stormwater management questions. All but five of the projects were completed after 1993. This is after the time priority watershed plans had been prepared for most of the major metropolitan areas in Wisconsin. More recent priority watershed projects have used the results of the later special monitoring projects.

All together, the special monitoring projects cost about $2.5 million. These costs were shared by the WDNR, EPA, and local units of government. Between five and eight projects are completed for each of questions 3, 4, 6, and 7. Our difficulties in selecting goals for reducing pollutant loads and changing other factors, such as flow volumes, is reflected in the fact that data is available for only two projects related to question 5. A report is available for all of the completed special monitoring projects. Since biological monitoring is done in every watershed project for questions 1 and 2, there are no special projects completed for these questions.

Role of Toxic Pollutants in Urban Streams - Question 3

Five special urban monitoring projects are completed that help characterize the impact of potentially toxic pollutants on the biological integrity of an urban stream. Three of the projects evaluated the toxicity of stormwater in Lincoln Creek. A total of 316 laboratory toxicity tests were performed on stormwater and baseflow samples with *Ceriodaphnia dubia, Daphnia magna,* and *Pimephales promelas.* No short term, 48-96-hour acute or 7-day chronic toxic effects, which could be solely attributed to stormwater runoff, were identified with the three laboratory test species (Ramcheck, 1995). Subsequent toxicity tests were modified to include longer-term *in situ* tests. Tests with *D. magna* performed in flow-through aquaria showed significant increases in mortality for 93% of the tests after 14 days of exposure (Crunkilton, 1996). Longer exposures of 17 to 61 days, with juvenile and adult *P. promelas* exhibited significant increases in mortality ranging from 30 to 95%. It appears that conventional wastewater effluent toxicity tests lack the sensitivity to detect the biological degradation observed in Lincoln Creek. The long-term *in situ* toxicity tests should be used for future special monitoring projects evaluating the toxicity of stormwater.

An *in vitro* bioassay with PLHC-1 (*Poeciliopsis lucida*) fish hepatoma cells was used to assess potential toxic potency of aryl hydrocarbon receptor (AhR) - active com-

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pounds, collected by semipermeable membrane devices (SPMDs) exposed to Lincoln Creek water (Villeneuve,1997). Dialysates from SPMDs exposed to Lincoln Creek water caused marked cytochrome P4501A induction in PLHC-1. SPMDs exposed to baseflow had consistently lower potencies than those exposed to high flows. Emperical evidence suggests that AhR-active polycyclic aromatic hydrocarbons (PAHs) can account for about 20 to 50% of the potency observed.

Monitoring of several urban streams in Milwaukee County, showed that the urban streams are highly degraded (Masterson, 1994). Stormwater discharges are blamed for high concentrations of pollutants in the water and bottom sediments, flashy flows, poor habitat, low diversity of aquatic organisms, and accumulation of pollutants in fish and crayfish tissue. A reference site was used to determine the degree of degradation. Water quality data compiled from four stormwater monitoring projects showed the concentrations of many potentially toxic pollutants are high enough to say that stormwater might be contributing to the degradation of the urban streams (Bannerman, 1996). All these findings describe the complexity of developing a solution to problems caused by stormwater.

Sources of Pollutants - Question 4

Results from six special urban monitoring projects are available to help determine the sources of stormwater pollutants. All but one of these projects provides data on the concentrations of pollutants in the runoff from different urban source areas. New sampling equipment was developed to collect sheet-flow runoff samples from roofs, parking lots, driveways, streets, industrial yards, and lawns. Source areas were sampled in residential, industrial, and commercial landuses. The relative importance of the pollutant load from each source area varies by pollutant and landuse. Study sites in Madison, WI, and Marquette, MI, showed streets as an important source for most pollutants and landuses (Bannerman, 1993; Burnhart, 1993; Waschbusch, 1998; Steuer, 1997). Lawns are an important source of phosphorus for all the study sites, while roofs contribute a relatively large amount of zinc in commercial and industrial landuses for the Madison study sites. Parking lots at the Marquette study site are contributing the largest amount of PAHs.

Not only the results from these projects identify important source areas for the study sites, but the data from these projects is also being used to calibrate SLAMM. This will increase our confidence in source area loadings determined for future priority watershed plans. Data from these projects are also helping us identify the activities responsible for depositing the pollutants on the different urban surfaces. For example, phosphorus concentrations in the runoff from streets increased with the greater percent tree canopy over the street (Waschbusch, 1998). This information will be used to make the model more sensitive to the tree canopy variations around a city. To more accurately model the runoff from lawns, runoff parameters were measured using a rainfall simulator on 20 Madison lawns (Legg,1996). Rainfall-runoff relations vary substantially between lawns, while the lawns that have been established less than three years produce much higher runoff volumes than older lawns.

An important number in every priority watershed plan is the comparison of agricultural and urban contributions of phosphorus and sediment to a stream. Stream phosphorus and sediment loads compiled from watersheds around the state indicated that the phosphorus unit-area loads in the southeast part of the state are similar for agricultural and urban landuses (283 and 318 lbs/sq. mi., respectively) (Corsi, 1997). Sediment unit-area loads are three times higher for the urban areas. A simple calculation with these numbers will be used to determine the importance of controlling urban sources of phosphorus and sediment.

Pollutant Reduction Goals - Question 5

Question 5 is probably the most challenging of the questions to answer, but it has received the least amount of attention. An inadequate answer to this question can greatly lower confidence in the suggested solutions to the stormwater problems. An interim method for predicting pollutant reduction goals is to combine the output of SLAMM with a probabilistic dilution model developed by the EPA (Corsi, 1995). A test of the method in Lincoln Creek demonstrated a reasonable agreement between the median measured and predicted event mean suspended solids concentrations. Pollutant loading reduction goals can be determined by reducing the pollutant loading output from SLAMM until the median event mean concentration in the stream is below the water quality standard. SLAMM loads are reduced by simply specifying a control in the model run.

The approach that will eventually replace using a probabilistic dilution model will be based on understanding the relationships between urban landuse activities and the conditions in the streams. An investigation of 103 streams in Wisconsin showed that a high amount of urban land use in a watershed is strongly associated with poor biotic integrity and weakly but significantly associated with poor habitat quality (Wang, 1997). There seemed to be a threshold value of the urbanization between 10 and 20% beyond which IBI values are consistently low. Performance standards based on observed threshold values can become the basis for setting pollutant and water volume reduction goals for urban streams.

Selection of Best Management Practices - Question 6

Having eight of the special monitoring projects, the study of best management practices has received the most attention. Monitoring data is available on the pollutant removal effectiveness for a wet detention pond, a multi-chamber treatment train, a Stormceptor, and street sweeping. A model was developed to test the removal effectiveness of infiltration devices. All of these types of practices are being used in Wisconsin. Two other projects summarized the

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costs of implementing different best management practices and the methods for monitoring industrial sites.

More monitoring data appears to be available for wet detention basins than any other type of practice. They are probably the most commonly used structural practice in Wisconsin. Results from monitoring a wet detention basin in Madison, indicate that a well-designed basin should remove about 90% of the solids, 50% of phosphorus, and 60% of heavy metals (House, 1993). The pond's sediment and associated pollutant removal efficiencies are both influenced by influent particle size distributions (Greb, 1997). Concentration data collected at the outlet occasionally exceeded the acute toxicity standards for zinc and copper. Toxicity testing on a pilot-scale wet detention basin indicated that toxic reduction goals might not be achieved by just using basins (Kron, 1998). Mortality for *P. Promelas* exposed to the treated Lincoln Creek stormwater was significantly reduced in only one of four test periods.

Evaluations of a multi-chamber treatment tank and a Stormceptor installed at city maintenance yards revealed very different pollutant removal efficiencies. The multi-chamber treatment tank achieved levels of control for many constituents of between 80 and 95%, while the efficiencies for the same constituents in the Stormceptor ranged from 20 to 30% (Greb, 1998). Both devices will easily retrofit into most land uses.

The water quality benefits of using mechanical street sweepers was evaluated at four paired test sites in Milwaukee County. Models developed during the project were used to determine street sweeping efficiencies for different times of the year. Street sweeping is most effective in the early spring during the heaviest street loads of the year and in the fall following leaf fall (Bannerman, 1983). Limited benefits are expected from any intensive sweeping program the rest of the year. Newer high efficiency sweepers are expected to perform better than the mechanical sweepers used in this study (Sutherland, 1997).

Although infiltration devices are rarely retrofitted in Wisconsin, they might be needed to some degree to fully accomplish our pollutant and water volume reduction goals. One concern about using infiltration as a practice is the potential threat to groundwater quality. A method for determining the potential mobility of 32 organic and seven inorganic pollutants during the infiltration of stormwater was developed (Armstrong, 1992). The main variables affecting leaching of stormwater pollutants are soil type selection, depth to groundwater, and water loading rate. Under high loading rates, a few meters of soil will probably not provide adequate protection of the groundwater. Inorganic pollutants (mostly metals) are less mobile than organic compounds (pesticides and PAHs). The calculated residence times per meter of soil for organic chemicals in a hypothetical infiltration system range from 15 days or less for "mobile" compounds to over 1,000 years for "very low mobility" compounds. These calculations assume a high water loading rate to the infiltration device. Predicted residence times for inorganic pollutants in a 1.0 meter soil layer subjected to a water infiltration rate of 60 meters per year range from less than 1 year for chromium to over 100 years for lead.

Costs of stormwater practices are sometimes difficult to estimate because not many of some types of practices have been installed in Wisconsin, and the costs of the ones that have been installed are not well documented. Not knowing the costs of the practices makes it very difficult to select cost-effective management alternatives for established urban areas. Some estimates of the capital and annual operation and maintenance costs are available for a number of practices (SEWRPC, 1991).

Different sampling methods are being used to test the effectiveness of stormwater practices designed to improve the quality of runoff from industrial sites. Evaluation of the effectiveness of the industrial practices will be difficult unless more is known about how the sampling methods can affect the interpretation of the data. Five different monitoring methods were tested at five different industrial sites (Roa-Espinosa, 1994). These five methods were (1) flow weighted composite, (2) time discrete, (3) time composite, (4) source area, and (5) first 30 minutes. Assuming that sampling at the outfall is the most representative sample, then time composite sampling is the best method. However, a new type of electronic source area sampler could make source area sampling a better choice, because the samples are collected closer to the source of contamination.

Results from testing the effectiveness of different best management practices is used to calibrate SLAMM. Practices not available in SLAMM, such as the multi-chamber treatment tank, are added to the model. The new effectiveness data will also be used to update the information about each practice in Wisconsin's stormwater manual (WDNR, 1994). Average long-term rainfall conditions for several regions of the state are used to run the model (Corsi, 1996).

Evaluation Monitoring - Question 7

The ability of the stormwater best management practices to achieve the designated uses in a stream is being determined for Lincoln Creek and the Menomonee River in Milwaukee County. Frequent chemical, biological, and physical monitoring being done for both streams. Plans are to continue the monitoring until implementation of the priority watershed projects is completed. Results from these two streams will be extrapolated to other urban streams. At a cost of about $40,000 per year for each stream this kind of intensive monitoring cannot be accomplished in all the priority watershed projects. All of the pre-practice installation monitoring is done for both streams. Results of the pre-practice installation monitoring have clearly documented the degradation of the water quality and biology in both streams (Wang, 1996; Owens, 1997). Several common statistical techniques have been tested to detect changes in the water chemistry data (Walker, 1993). The

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application of non-parametric tests to regression residuals for storm load data appears to be the best approach for estimating minimum detectable change for a known or estimated "before" condition.

To help quantify the changes in the stream, a version of the Index of Biotic Integrity (IBI) was developed for warm water streams in Wisconsin (Lyons, 1992). Guidelines were developed for evaluating fish habitat in Wisconsin streams (Simonson, 1994).

Future Urban Monitoring Plans

Results from the completed special monitoring projects greatly increased the amount of information available to answer the seven stormwater management questions. In any new plan, however, we could not totally defend the answers to the seven stormwater management questions; if lead is targeted as a pollutant to control, for example, we still could not totally defend its role in the degraded biology or the levels of lead reduction suggested in the plan. Without good monitoring data, each implementation effort is to some degree an experiment, whose results will probably not be known until it is too late to make any major adjustments to the types of best management practices implemented.

Having the right kind of monitoring data can also influence people's acceptance of the solutions offered in a priority watershed plan. Whenever municipalities, industries, and others cooperating in the stormwater clean-up effort have some doubts about the answers to the management questions, it diminishes the chances of completely implementing the priority watershed plan.

Future Products

Our experience with finding answers to the seven stormwater management questions gave us some ideas on the type of additional information we need to improve our answers. We identified eight products we need to develop using special monitoring projects (Table 3). All the products are important, but developing biological criteria and stormwater performance standards for urban streams would probably give the biggest boost to the credibility of our answers. We would like to set a goal of having all of the products over the next five years. Realistically, it will probably take longer to develop stormwater performance standards.

Biological criteria are needed to quantify a potential use of every stream. This should be less subjective than the stream classification procedures we follow now to determine the uses of a stream. A set of indices, such as the IBI and the HBI, would identify the potential use of the urban streams in every Wisconsin sub-ecoregion. Development of the criteria would be a five-year effort requiring the collection of data in both rural and urban streams. Some of the data needed is already in WDNR files.

Closely related to the biological criteria is the identification of the pollutants causing a toxic response in urban streams. Toxic pollutants could be a limiting factor in the selection of best management practices. Although a toxic effect has already been identified in one urban stream, it is not known which pollutants are responsible for the observed toxicity or what degree of urbanization is required to cause a toxic response. Using methods already developed, it would probably take about three years to develop an understanding of which pollutants are toxic and how the amount of urbanization effects their toxicity.

Table 3. Products to be Developed using Future Special Urban Monitoring Projects

Question No.	Products
1	Biological Criteria for urban streams.
2	None
3	Method to identify which toxic pollutants are important in each stream and what levels of control are needed.
4	SLAMM calibrated for all source areas and all the problem pollutants.
5	Stormwater performance standards designed to achieve biological criteria - standards based on % connected imperviousness, flow rates and volume, D.O. levels, buffers, pollutant loadings, and temperature.
6	Method of selecting most cost-effective practices to achieve performance standards. SLAMM capable of testing all practices. Stormwater manual describing suggested management alternatives for the most commonly occurring land use mixtures.
7	Location to showcase benefits of stormwater management.

SLAMM is calibrated for many of the pollutants washed off many of the source areas. But more calibration is needed for the toxic pollutants in runoff from some of the source areas, especially gas stations, parking lots, and industrial paved surfaces. A three-year monitoring effort using our existing source area monitoring methods would produce the numbers to finish the calibration of SLAMM.

A performance standard is a threshold value for a biological, chemical, or physical factor that, if achieved, will help meet biological criteria for a stream. The threshold values are for the factors that affect the biological integrity of any stream. At least seven types of performance standards need to be developed to meet the biological criteria or improve the biological integrity of a stream. They include maximum temperatures, minimum dissolved oxygen levels, minimum and maximum flows, maximum water volumes, types of riparian vegetative buffers, annual sediment loading, and the combined annual loading of problem toxic pollutants. Percent connected imperviousness is also included as a factor because it is a good way of combining the effects of all factors without having to understand the effect of each one. Target values for all of these factors would be the basis for developing manage-

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ment alternatives. These target values will probably vary between sub-ecoregions. The difference between the performance standards for a stream and the existing values for these factors determines the goals for reducing pollutants or changing other factors.

A multi-variate statistical analysis is going to be done to determine the importance of each one of these factors. A great deal is already known about threshold values for some of the factors, especially flow, temperature, and dissolved oxygen (Raleigh, 1986). Most of the monitoring over the next five years will be designed to better understand the threshold values for percent connected imperviousness and pollutant loadings. Work has already started on the percent connected imperviousness factor with the collection of biological data from 45 streams with different degrees of urbanization. A less expensive method is being developed to estimate annual pollutant loadings. Data collection has started on a project to calibrate a model designed to predict stream temperature changes during a runoff event in an urban area.

Another important product is the development of cost-effectiveness curves for different management alternatives. Cost-effectiveness would be based on pollutant removal relative to different annual costs of the alternatives. Usually, a combination of practices would be included in each alternative. Curves might vary by land use and/or type of pollutants being controlled. The curves would identify the least expensive alternative for the level of control desired.

All the most promising best management practices will be tested in Wisconsin. A special emphasis will be put on infiltration and filtration devices. We will also try to document the water quality benefits of educating the public on stormwater management. All the results of these efforts will be used to calibrate SLAMM and update our stormwater management manual.

Every environmental management program needs some place to showcase the benefits of their efforts. This will be essential to justifying the long-term funding commitments required by municipalities, industries, and other groups responsible for stormwater management. Although evaluation monitoring has already began at Lincoln Creek and the Menomonee River, at least one more site is needed. We are looking for a site where a good quality stream in an urbanizing area could be saved by the proper use of best management practices.

Future Types of Special Urban Monitoring Projects

A lot of monitoring is going to be required to produce all eight products. It is difficult at this time to describe all the types of special monitoring projects that will be needed to develop the eight products, but it is useful to suggest a list of projects. About 28 special monitoring projects should provide enough information to develop the products we need (Table 4). Completion of all these projects will require at least five years or more to complete for a cost of at least $4.5 million. Work has already started on five of these projects. Most of the other projects are just at the suggestion stage.

Table 4. Future Special Urban Monitoring Projects

Question No.	Monitoring Projects
1	a. Collect biological data at test and reference sites in all sub-ecoregions. b. Develop IBI for small warm water streams.
2	None
3	a. Test response of organisms to different toxics in stormwater. b. Test response of organisms to serial dilutions of problem toxics. c. Test toxic response in streams with different % connected imperviousness.
4	a. Test electronic sheet flow sampler. b. Collect runoff from all source areas in three WI ecoregions. c. Measure runoff coefficients for lawns in three WI ecoregions. d. Evaluate relationship between lawn characteristics and amount of runoff. e. Measure pollutant concentrations for streets with different traffic volumes. f. Measure street phosphorus levels for streets with different tree canopy. g. Measure accumulation and washoff functions for street solids. h. Measure pollutant loadings during snowmelt.
5	a. Calculate amount of infiltration needed to maintain normal baseflows. b. Measure effect of excess runoff volumes on fish habitat. c. Collect all types of data in streams with different degrees of urbanization. d. Calibrate temperature model. e. Evaluate importance of flow, habitat, chemistry to quality of stream. f. Evaluate importance of different buffer widths to stream quality. g. Determine how many grab samples are need to estimate annual loading.
6	a. Measure effectiveness of two infiltration devices. b. Measure effectiveness of high efficiency sweepers. c. Measure effectiveness of two filtration devices. d. Measure benefits of public education. e. Summarize cost of building and maintaining all types of practices. f. Calibrate selected flow model. g. Test controls by using historical and new fish data from urbanizing areas.
7	a. Measure use changes in urban and urbanizing streams.

Projects already started include (1) development of IBI for small warm water streams, (2) determining the relationship between percent connected imperviousness and stream quality indicators, (3) calibration of a model to predict temperature changes in an urban stream during a runoff event, (4) developing a less expensive method of deter-

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mining annual pollutant loads, and (5) evaluating benefits of implementing practices in urban streams.

Summary

A combination of some monitoring in every urban stream and special urban monitoring projects is helping Wisconsin retrofit urban best management practices that achieve the designated uses of the streams for as little cost as possible. All the monitoring activities have been designed to answer seven stormwater management questions. Each question relates to a type of information needed to complete one or more chapters in a priority watershed plan prepared for Wisconsin's Nonpoint Source Program. Biological monitoring done in every urban stream identifies the designated uses and the reasons for any of the observed problems. Special urban monitoring projects enhance the answers to the first three questions, such as identifying the potentially toxic pollutants, and provide answers to the last four questions. Results of special projects to determine the sources of pollutants and the effectiveness of best management practices are most used results in the priority watershed plans completed over the last ten years.

Results are available from 26 special urban monitoring projects completed over the last 17 years for a cost of about $2.5 million. Most of these projects were completed after 1993. These results provide excellent answers to parts of larger problems; others parts of those problems remain unasked or unanswered. More information is especially needed to determine goals for each priority watershed project and to determine the best management alternatives. Although it is important to continue the efforts to retrofit urban areas with what we know, it is also important to lower the risk in making future management decisions by conducting additional monitoring projects.

Twenty-eight new special urban monitoring projects resulting in eight essential products are recommended. Development of biological criteria and performance standards for urban streams are two of the products essential to the success of future stormwater management efforts. Work has already started on five new special monitoring projects. These projects will help define the use of "percent connected imperviousness" as a performance standard and provide a method for predicting the changes in stream temperature during a runoff event. Another project will document the changes in two urban streams during and after the implementation of best management practices. Municipalities, industries, and other groups cooperating in the stormwater management efforts will be able to use the results of these and other new projects to improve their confidence in management decisions that could cost millions of dollars.

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Considerations and Approaches for Monitoring the Effectiveness of Urban BMPs

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The purposes of this paper are to 1) describe some of the problems with typical Best Management Practice (BMP) monitoring and effectiveness reporting and to 2) suggest the utilization of consistent stormwater monitoring techniques. This will allow the data collected on the effectiveness of individual best management practices (BMPs), including retrofit BMPs, to be useful for a particular site, and to also be useful for comparing studies of similar and different types of BMPs in other locations. Many BMP effectiveness studies in the past have provided only limited data useful for assessing BMP design and selection on a wide scale. This paper overviews some of the problems of past BMP effectiveness studies from the perspective of comparability between studies. It suggests some of the ways that data could be collected to make it more useful for assessing factors (such as settling characteristics of inflow solids and physical features of the BMP) that might have led to the performance levels achieved. Finally, it also discusses other considerations that affect data transferability, such as effectiveness estimations, statistical testing, etc.

Introduction

Many studies have been completed which have assessed the ability of stormwater treatment BMPs (e.g., wet ponds, grass swales, stormwater wetlands, sand filters, dry detention, etc.) to reduce pollutant concentrations and loadings. However, in attempting to summarize the information gathered from these individual BMP evaluations it is very apparent that inconsistent study methods and reporting make wider scale assessments difficult. For example, individual studies often included the analysis of different constituents and utilized different methods for data collection and analysis. These differences alone contribute significantly to the range of BMP effectiveness reported. This makes assessing what other factors may have contributed to the variation in performance almost impossible.

In one review of the use of wetlands for stormwater pollution control (Strecker et al., 1992), a summary of the literature on performance of wetland systems and the factors that may have led to the reported pollutant removals was prepared. The literature was inconsistent with respect to the constituents analyzed and the methods used to gather and analyze data. A number of pieces of information, if collected and recorded, would have improved the ability to evaluate the effectiveness of stormwater wetlands as BMPs and facilitated the transfer of that knowledge into better design practices. Urbonas (1994 and 1995) and Strecker (1994) summarized the information that should be recorded about the physical, climatic, and geological parameters which likely affect the performance of a BMP, and considerations regarding sampling and analysis methods. This paper presents 1) a suggested list of constituents for analysis along with recommendations for reporting data, 2) methods of reporting pollutant removal efficiencies, 3) a brief discussion of statistical approaches to selecting the number of samples needed, 4) methods for including detection limit data, 5) sample collection considerations, and 6) the need for dry weather assessments.

BMP Performance Study Inconsistencies

Studies of BMP effectiveness have utilized significantly different:

- Sample collection techniques (e.g., from sample collection types (grab, composite, etc.), flow measurement techniques, to how the sample was composited, etc.);
- Constituents, including: chemical species, methods (detection limits), form (e.g., dissolved vs. total, vs. total recoverable, etc.), and treatment potential;
- Data reporting on tributary watershed and BMP design characteristics (e.g., tributary area or watershed attributes such as percent impervious, land use categories, rainfall statistics, etc.);
- Effectiveness estimation (at least four techniques have been utilized to assess effectiveness which can cause significant differences in pollutant removal reporting, with the same set of data), and potential alternatives to reporting just concentration/loading reductions; and
- Statistical validation of results (typical lack of statistical tests to determine if the reported removal efficiency

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can in fact be shown to be statistically different than zero).

Any of the above topics would require an in-depth discussion beyond the scope of this paper to fully explain. Therefore, this paper will present a brief overview of each of these and some potential solutions to improving how data is collected. EPA together with ASCE is currently developing a set of protocols and a database on BMP performance studies with the purpose of improving the consistency of BMP monitoring information. This project includes:

- Developing Protocols for BMP Monitoring
- Conducting an evaluation of existing information to assist the EPA Wet Weather FACA and contribute to EPA's Stormwater Toolbox (as identified in Draft Phase II Stormwater Regulation Preamble)
- Developing a data base on BMP performance studies

The overall goal is to improve the BMP effectiveness information base to:

- Develop information to improve designs
- Improve performance information

The data base specifies a chosen set of reporting information, but does not tell how to develop such information. For example, it does not specify what a flow-weighted composite sample is and how it should be collected. The next step beyond the EPA protocols and data base effort should be a guidance document on monitoring data collection strategies and techniques to improve their consistency and transferability. It should be recognized that with the development of the database and the protocols, it will be a number of years (5 to 10) before significant new studies on BMPs are conducted utilizing the protocols to allow for a more rigorous evaluation of BMP selection and design factors.

Sample Collection Techniques

The differences among sample collection techniques alone is enough to make comparing different studies questionable. These include differences among how flows are measured to how samples are composited to formulate an "event mean concentration." Some studies have utilized grab samples, and the results of these studies in evaluating BMP performance are limited. Typically studies will include the collection of flow-weighted composite samples (either automated or hand collected). These studies involve various techniques (often not reported very well) for measuring flows. The flow measurements themselves are subject to a large variation.

The Federal Highway Administration is currently conducting a study of monitoring techniques for characterizing stormwater runoff hydrology and water quality from highways. The study, being completed by Woodward-Clyde, included a component conducted by the USGS (Waschbusch and Owens, 1998) which addressed the potential differences in flow measurement techniques in a pipe system in Madison, WI. An in-depth dye-dilution method was utilized to calibrate a Palmer-Bowlus flume with a bubbler pressure measurement. The study evaluated 23 flow measurement techniques including commercially available packages and individual component systems.

Figure 1 is a summary of the results of flow measurements, showing the average percent differences from the calibrated flume. These data summarize 50 storm events which were measured over a 6-month period. As the figure demonstrates, the error in flow measurements is easily on the order of plus or minus 25% over a range of storms. The flow measurements for individual storms varied even more. If samples are composited based upon flows (either using automated or using grab samples), they are subject to an error in collection times (for automated systems) or in composited amounts (grab sample composited) and therefore could result in errors in estimates of event mean concentrations (especially for constituents which vary over the course of a storm event). It should be strongly noted that these results are for one site only and should not be interpreted as indicative of how any particular system identified might perform at another site. It is imperative that researchers thoroughly evaluate potential flow measurement alternatives and implement the method that will result in the best information possible.

Another aspect of the study addressed how many samples should be collected to compile a "flow-weighted" composite sample. Figure 2 demonstrates the large variability in sampler bottle configurations. These configurations often drive researchers into selecting the number of "grab" composite samples to collect. For example, in the NPDES monitoring for Texas (Brush, et al., 1994), the chosen strategy was to collect one sample into each bottle of the 8-bottle configuration (this was successful if it rained sufficiently). In the Portland and Eugene NPDES Sampling (WCC 1993a and WCC 1993b), an attempt was made to collect 24 "grab" samples during the course of an event. Figure 3 shows a typical storm event from the Portland program and specifically the points at which a sample was collected. From the variability in flows observed, one can surmise the pollutant concentrations were also fluctuating extensively (later confirmed by within-storm sampling). Having only eight samples during this event may not have accurately characterized the event mean concentration (EMC). Collecting three times the samples to "construct" a flow-weighted sample would appear to reduce the chances of anomalies (variability) during a storm event influencing the overall estimate of the average concentration. Early results from our FHWA study indicate that one should attempt to collect at least 12 to 16 individual samples to form a composite sample.

The study also has evaluated the potential effects of sample lift (e.g. pumping up from underground or from stream bottoms) and has found that the newer samplers

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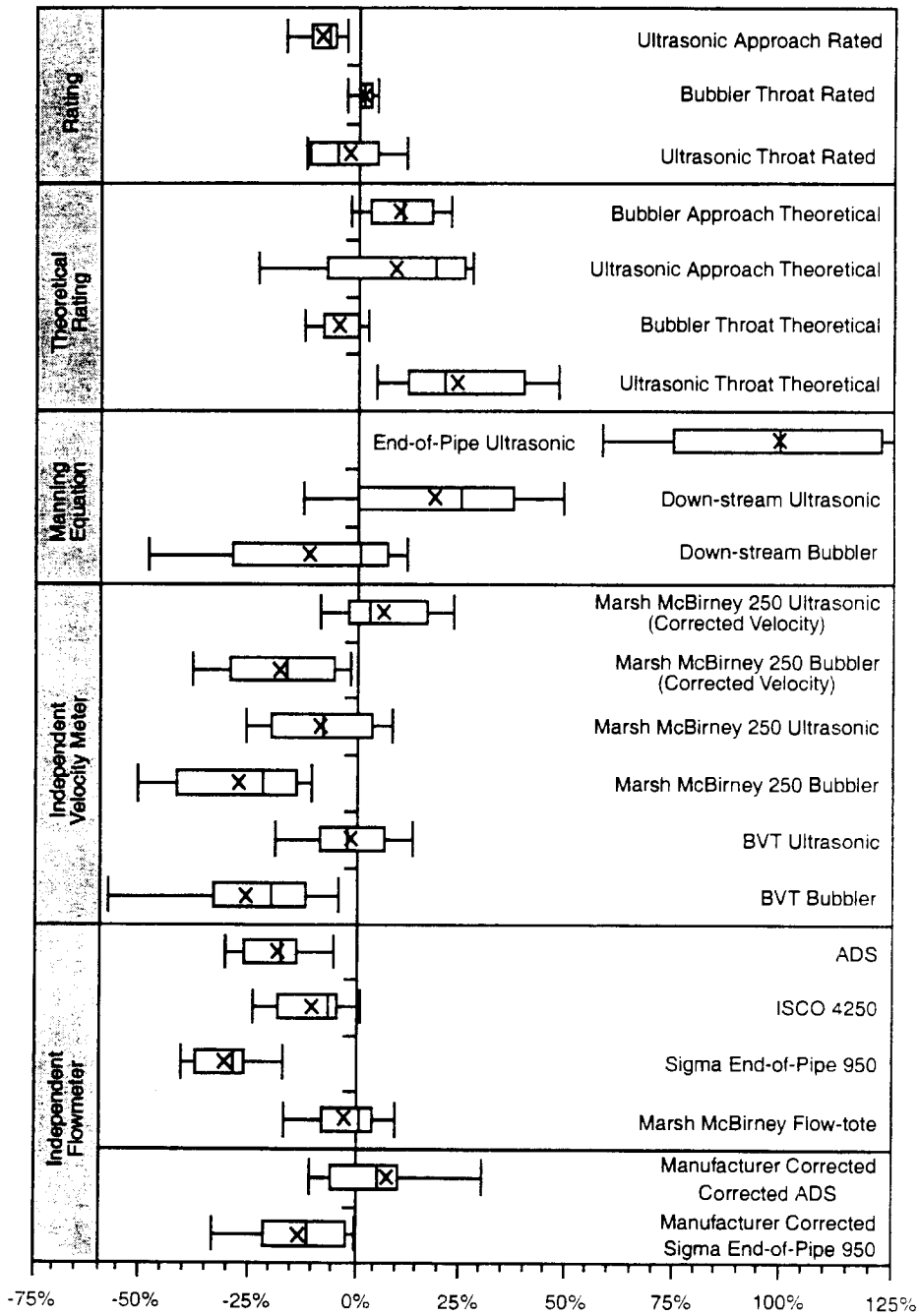

Figure 1. Boxplot of the percent differences between total storm volumes computed using various flow estimation methods and the total storm volume of the bubbler approach rated discharge (bold line at 0%). (Waschbusch and Owens, 1998).

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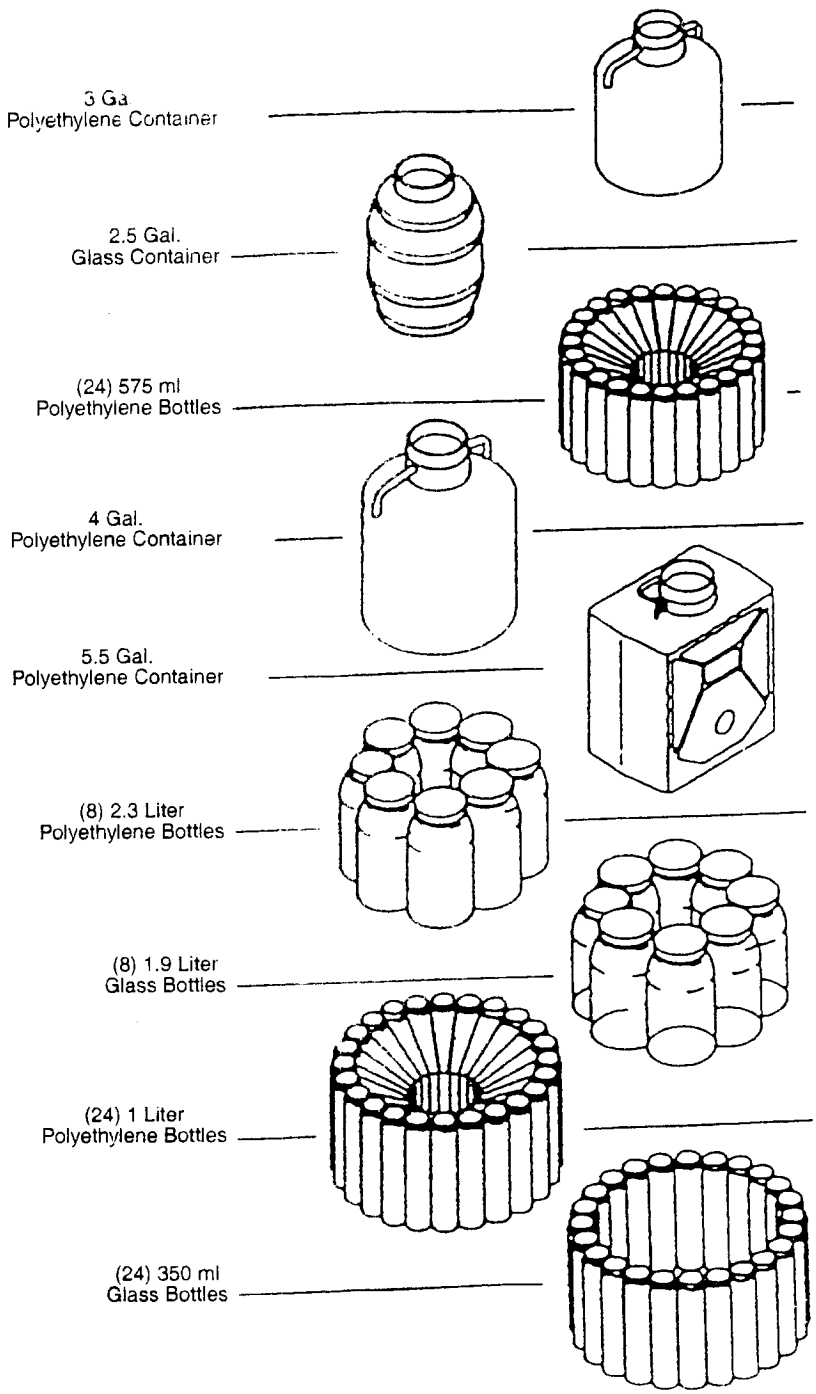

Figure 2. Typical automated sampling bottle configuration options.

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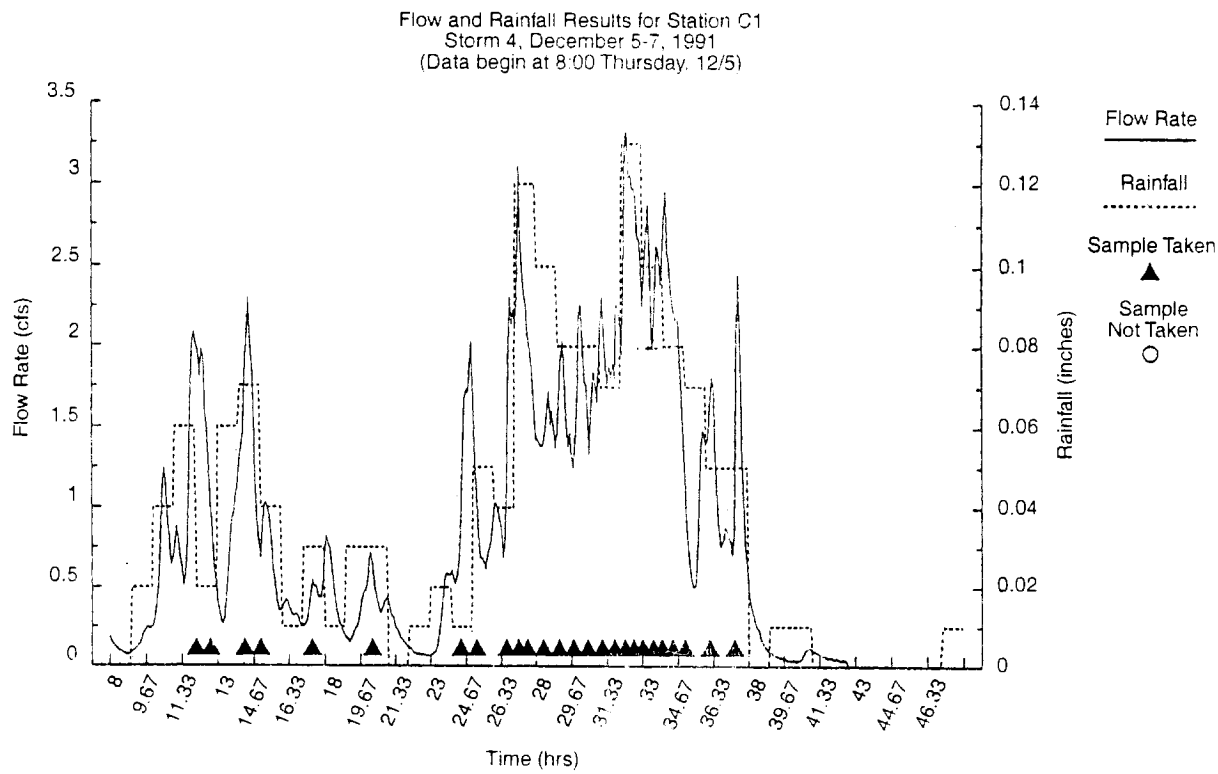

Figure 3. Typical hydrograph indicating measured rainfall, runoff, and water sample collection times from automated flow and water quality sampling for the Portland NPDES Stormwater Monitoring Program.

(with stronger pumps) do not appear to cause any separation of suspended solids as they are lifted up to 20 feet. At the end of the study, a guidance document on sampling of highway runoff will be developed. These are just some of the numerous differences in sampling methods that could lead to differences in results between BMP studies.

Constituents Assessed

A very wide variety of pollutants have been analyzed in both BMP studies and characterizations studies. The EPA protocols study has developed a recommended set of constituents for BMP testing programs. These were developed from the review of previous studies and an understanding of costs and likelihood of providing meaningful results. Below is a discussion of how these constituents were selected (adapted from Strecker, 1994).

Since NURP and prior to the Phase I Stormwater NPDES monitoring programs, there have been a number of studies which continued to assess pollutant concentrations in stormwater runoff. These included the Federal Highway Administration's highway runoff program (Driscoll et al., 1990) and some selected studies done in a few locations. These studies typically were not consistent with the standard NURP protocols. Based upon the 1987 amendments to the Clean Water Act, EPA required operators of municipal separate storm drainage systems that served populations of over 100,000 to collect flow-weighted composites at a minimum of five stations to characterize residential, commercial, and industrial runoff quality. Only a few additional parameters have been identified as "problems" in stormwater, based upon these post-NURP studies (this despite the improved analytical methods that have become available for conducting laboratory analyses). In addition, NURP focused primarily on residential and commercial land uses, while NPDES testing included industrial land uses which were suspected of having more pollutants present.

However, there has not been a comprehensive review by EPA or others of the newly collected stormwater information to assess the results of requiring the analysis of over 130 constituents, including priority pollutants. This type of review is needed. EPA's requirements included monitoring three storms at selected stations. This number of storms is only useful for identifying potential problem pollutants. Statistically, these are not enough data to perform

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a meaningful regional or other factor analyses of urban stormwater concentrations, although they could provide useful information on rates of detection. This analysis would be helpful in selecting constituents for BMP monitoring.

The choice of constituents to include as "standard pollutants" is a subjective one. As an example, some would argue that cost should be a primary consideration; others would say that it should not. In making the recommended list of monitoring constituents, the following characteristics were considered:

- The pollutant is prevalent in typical urban stormwater at concentrations that could cause water quality impairment.
- The analytical test can be related back to potential water quality impairment.
- Sampling methods for the pollutant are straightforward and reliable for a moderately careful investigator.
- Analysis of the pollutant is economical on a widespread basis.
- The pollutant is one for which treatment is a viable option.

Not all of the pollutants recommended fully meet all of the factors listed above; however, the factors were considered in the recommendations. When developing a list of pollutant analyses for an individual BMP evaluation, it is important to consider the upstream land use activities. The parameters recommended below are present and of concern in "typical" urban stormwater.

The Nationwide Urban Runoff Program (NURP) (EPA, 1983), which included monitoring of land use runoff and BMP performance at over 28 cities nationwide, adopted consistent data collection methods and analytical parameters. Results from the NURP program could be used to evaluate similarities and differences in pollutant concentrations in urban stormwater from different and similar land uses, and could be used to explain what might be causing these differences. The following pollutants were adopted by NURP as "standard pollutants characterizing urban runoff":

TSS	Total suspended solids
BOD	Biochemical oxygen demand
COD	Chemical oxygen demand
TP	Total Phosphorus
SP	Soluble phosphorus
TKN	Total Kjeldahl nitrogen (as N)
$NO_2 + NO_3$	Nitrate + nitrite (as N)
CU	Copper
PB	Lead
ZN	Zinc

Oil and grease was not included because of the difficulty in obtaining representative samples. On a less consistent basis, NURP also monitored for pollutants including other metals, dissolved metals, semi-volatile organics, volatile organics, pesticides, and herbicides.

Presented below is a brief discussion, by group, of the pollutants that are recommended to be included in a base list, then several that may occasionally be recommended.

Total Suspended Solids (TSS). The term "suspended solids" is descriptive of the organic and inorganic particulate matter which is of a size and type that allows the particles to stay suspended in water. The solids load in a waterbody is influenced by a number of factors including but not limited to: particle sizes, stream flows, climate, geology, and vegetation of each drainage system. The conditions under which suspended solids are considered a pollutant is a matter of definition. In general, suspended solids are considered a pollutant when they significantly exceed natural concentrations and have a detrimental effect on water quality and/or beneficial uses of the water body.

Suspended sediments are often used as a surrogate for other contaminants which bind or adsorb easily with fine particulate matter, including heavy metals. Although TSS is often highly correlated with other parameters, it is generally not a strong enough correlation to eliminate the need to address other parameters specifically. Figure 4 shows the relationship between TSS and zinc for pooled stormwater runoff monitoring data from all ten stations monitored in Portland, Oregon for the NPDES program (WCC, 1993a) and from the seven stations that were from piped systems. Although the relationship is statistically significant (R^2 of .38 for piped stations), it does not explain a significant amount of the variability. Similar results were found for almost all other parameters. It should be noted that for individual stations, the relationships between TSS and many pollutants were sometimes much higher, but this would mean that one would have to monitor enough times to establish the relationship. Therefore, TSS does not appear to be a good predictor of other pollutants, without significant data collected from each station. However, TSS is one good indicator of pollutant removal efficiency (e.g., because of the tendency for many pollutants to be associated with fine particulates) and should be included in any evaluation of BMP performance.

Many BMPs rely on sedimentation as the primary pollutant removal mechanism. It is recommended that samples also be analyzed for some measure of the expected settling rate (treatment potential) of TSS. The performance of a BMP that relies on sedimentation and even filtering can be greatly affected by the particle sizes and densities present in the influent. If the influent TSS is characterized by very small particle sizes, and therefore slow settling velocities, it will be much more difficult to treat. The settleability of influent solids has not been adequately addressed in performance comparisons, and may be one of the significant reasons that measured performance varies so highly from similar BMP to BMP.

For consideration, the particle size distribution in street dirt found in Sartor and Boyd (1972), as shown in Table 1,

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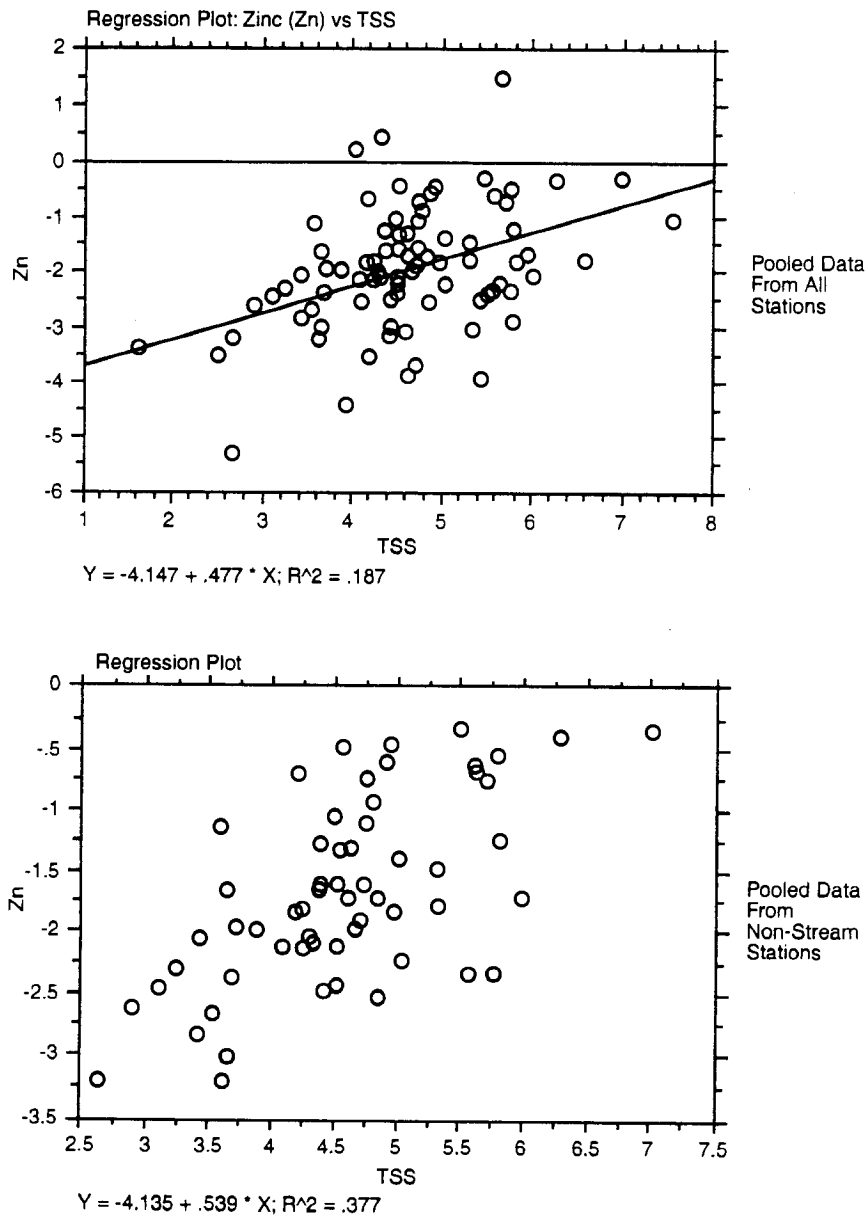

Figure 4. Natural logarithm regression plots of zinc (Zn as ln(mg/l)) vs. total suspended solids (TSS) for pooled Portland, OR stormwater monitoring event mean concentration data. (Woodward-Clyde Consultants, 1993a)

might be an appropriate gauge of the "treatment potential" of stormwater. As Table 1 indicates, these distributions vary considerably from city to city and likely from site to site. One can easily surmise that if testing were performed on similar catchments and BMP designs, that there could be a large difference in BMP performance results from these sites just due to the particle size differences alone. Another potential measure of the treatment potential would be information from settling column tests as those discussed by EPA in its manual on combined sewer overflow control (EPA, 1993a).

Oxygen Demand. Oxygen demand refers to the amount of oxygen that will be consumed by biological or chemical reactions involving organic compounds. The decomposition of biodegradable materials by natural soil and water bacteria draws upon the dissolved oxygen resources of a water body. This process is countered by natural re-aeration processes that occur in all water bodies to varying degrees. Significant reductions in dissolved oxygen concentrations can result when the demand rate exceeds the rate of replenishment through re-aeration. In general, moderately high dissolved oxygen content is necessary for the maintenance of healthy aquatic ecosystems. The relationship of oxygen-consuming discharges to the amount of dissolved oxygen in a receiving water body, therefore, is fundamental to the maintenance of environmental quality in natural water bodies. However, the tests available for

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Table 1. Particle Size Fractions of Street Dirt from Selected Locations.

Size Ranges	Milwaukee	Bucyrus	Baltimore	Atlanta	Tulsa
>4800 μ	12.0%	–%	17.4%	–%	–%
2000 - 4800 μ	12.1	10.1	4.6	14.8	37.1
840 - 2000 μ	40.8	7.3	6.0	6.6	9.4
246 - 840 μ	20.4	20.9	22.3	30.9	16.7
104 - 246 μ	5.5	15.5	20.3	29.5	17.1
43 - 104 μ	1.3	20.3	11.5	10.1	12.0
30 - 43 μ	4.2	13.3	10.1	5.1	3.7
14 - 30 μ	2.0	7.9	4.4	1.8	3.0
4 - 14 μ	1.2	4.7	2.6	0.9	0.9
>4 μ	0.5	–	0.9	0.3	0.1
Sand %, 43 - 3800 μ	92.1	74.1	82.1	91.9	92.3
Silt %, 4 - 43 μ	7.4	25.9	17.1	7.8	7.6
Clay %, <4 μ	0.5	–	0.9	0.3	0.1

Note: μ = microns
Source: Sartor and Boyd, 1972

assessing oxygen demand are not straightforward indicators of potential problems.

Biochemical Oxygen Demand (BOD). The 5-day BOD test provides an indirect measure of the quantity of biologically degradable organic matter in water in terms of the amount of oxygen required by microorganisms to oxidize it to carbon dioxide and water. The BOD test is quite variable. A number of factors can affect results, including the quality of the seed culture utilized in the test. The BOD test can also be inhibited by toxicants in the sample, which may react differently once the runoff mixes with the receiving water. The levels of BOD that are normally found in urban stormwater are near detection limits for the BOD test. Therefore, they are subject to wide variation. Therefore BOD has not been recommended as a parameter. Instead, TOC (Total Organic Carbon) has been identified as a more consistent measure of available organic material, which could be contributing to oxygen demand.

Chemical Oxygen Demand (COD). The COD test provides a more rapid and consistent measure of oxygen demand than BOD tests. The consumption of oxygen from an introduced strongly oxidizing chemical agent is measured by this test. As a result, it typically measures appreciably higher levels of oxygen demand than will be produced by biological decomposition because it oxidizes some organic compounds that are not biodegradable, and may also react with inorganic compounds as well. In urban stormwater, for example, COD levels are typically found to be about 8 to 10 times greater than BOD levels. COD measures a "maximum possible," but not probable, oxygen demand.

Nutrients. Nutrients are necessary for the growth and support of biota in natural water systems. Excessive quantities can result in the over-stimulation of biological growth and the creation of objectionable water quality conditions (eutrophication). Some forms of nutrients can also be toxic (e.g., ammonia). In general, the most important nutrient factors causing an acceleration in algal production are nitrogen compounds and phosphorus.

Nitrogen. Nonpoint sources of nitrogen include lawn fertilizers, leachate from waste disposal in dumps or sanitary landfills, atmospheric fallout, nitrite discharges from automobile exhausts and other combustion processes, natural sources such as mineralization of soil organic matter, and farm-site fertilizers and animal wastes. Many water treatment methods have no significant effect on nitrate removal from water (Dunne and Leopold, 1978).

Three forms of nitrogen have been analyzed extensively in stormwater runoff water quality studies. These are nitrite plus nitrate (NO_2 + NO_3), ammonia nitrogen (NH_3), and total Kjeldahl nitrogen (TKN). The latter, named after the analytical test procedure, provides a measure of ammonia and organic nitrogen forms that are present. The first (NO_2 + NO_3) provides a measure of the inorganic nitrogen. There is usually very little nitrite in stormwater. Nitrate (NO_3) is very mobile and is usually difficult to treat utilizing stormwater BMPs. Ammonia nitrogen can be toxic to aquatic life. It can be assessed for toxicity to aquatic life with data on pH and temperature. The inorganic (NO_2 + NO_3) and ammonia nitrogen are recommended. All forms should be reported as mass of nitrogen (N).

Phosphorus. Phosphorus is used by algae and higher aquatic plants and may be stored in excess of use within plant cells. With decomposition of plant cells, some phosphorus may be released immediately through bacterial action for recycling within the biotic community, while the remainder may be deposited with sediments.

R0022402

Phosphorus enters waterways from many of the same sources as nitrogen. Domestic sewage contains significant concentrations of phosphorus which are contributed by detergents and human wastes. Primary and secondary treatment processes normally remove only about 20 to 30% of this element from sewage (Dunne and Leopold, 1978). Fertilizers and the erosion of soils rich in phosphorus can also be a potential source.

Three forms of phosphorus have been somewhat routinely analyzed in stormwater runoff studies. These include total phosphorus (TP), soluble phosphorus (SP), and ortho-phosphate (OP). Ortho-phosphate indicates the phosphorus that is most immediately biologically available. Soluble phosphorus includes both the ortho-phosphate and a fraction of the organic phosphorus. Most all of the SP is usually OP, however. Total phosphorus includes phosphorus in the forms that may not be as readily biologically available plus the forms discussed above. TP and OP are recommended for inclusion in a monitoring program, as they characterize both the total and bioavailable forms of phosphorus. All forms should be reported as mass of phosphorus (P).

Metals. Heavy metals such as copper, lead, and zinc are naturally released in very small quantities by the weathering of exposed soils and mineral deposits, corroding metal surfaces, decomposing paints, and certain corrosion-control compounds. Heavy metals tend to have comparatively low solubilities and are often mobilized by forming soluble complexes with humic materials or by becoming attached to clay particles. Heavy metals have been consistently identified as the most significant toxics found in urban stormwater and often exceed water quality criteria for aquatic life.

These metals are present in the biosphere as trace elements and are micronutrients necessary for plant and animal growth. Heavy metals are of concern because elevated concentration levels of soluble forms in natural water bodies can produce toxic effects in biota. Sources include domestic and industrial point-source discharges, urban stormwater runoff, and direct atmospheric deposition. In this paper, copper (Cu), lead (Pb), zinc (Zn), and cadmium (Cd) have been recommended for inclusion in a monitoring program because stormwater runoff water quality studies conducted at many urban locations have indicated that these metals are almost always present, and are at concentrations which tend to be elevated, relative to other heavy metals. They also can be used as surrogates for other heavy metals, as they tend to display the range of transport characteristics for heavy metals. However, other heavy metals should be analyzed if there are known sources of significant quantities of these metals in influent flows.

It is recommended that both the total and dissolved form of each be analyzed. Based upon EPA's recommendation, the dissolved fraction should be compared to water quality criteria, with modifications to the criteria as noted in EPA (1993b). To compare data to criteria, hardness should be measured for each sample. Too often, metals data are compared to criteria using an average hardness value not directly associated with the monitoring, and not associated with storm events. In the Williamatte Valley of Oregon, stormwater sampling has shown that hardness values during storm events are quite low, which results in low criteria values.

Total concentrations are valuable in assessing the overall reduction of the heavy metal in both soluble and particulate forms. There is a concern about the long-term bioavailability of these metals in sediments and sediment standards are beginning to be developed and implemented.

When conducting these tests, it is recommended that low detection limits be achieved. For copper, lead, and zinc, the detection limit should be 1 μg/l and for cadmium 0.2 μg/l. This will minimize problems with analyses that include below detection limit data, which can severely impact performance evaluations. Special "clean" procedures will be necessary to achieve low detection limits, both in the laboratory and in the field.

Too often, BMP effectiveness for metals is estimated based upon data that is very near or below detection. This is troublesome when both the inflow and outflow concentrations are at or near detection, and effectiveness is based upon a storm-by-storm comparison of loads or concentrations. It is recommended that if both the influent and effluent concentration are within five times the method detection limit, the pollutant data pair not be considered in the effectiveness analysis if a storm-by-storm method is used. If statistical characterizations of the inflow and the outflow concentrations are utilized to assess effectiveness and some of the data are below detection, appropriate techniques should be utilized. Driscoll et al. (1990) describes a method to address detection limit data. The setting of below-detection values to 0 or 1/2 the detection limit or the detection limit, will typically lead to an underestimation of the mean.

Oil and Grease. Oil and grease is a prevalent constituent in urban runoff and often exceeds discharge limits set by states (such as 10 mg/l in Oregon for industrial stormwater permits). In a study of oil and grease concentrations in urban runoff in Richmond, California, Stenstrom et al. (1984) found that oil and grease concentrations in runoff from commercial properties and parking lots are about three times higher than from residential and open areas. The NURP program did not address oil and grease as a standard constituent. Accurately measuring oil and grease is very difficult due to its affinity for coating sampling bottles and sampling tubes and its highly non-uniform distribution in the water column (except in the most turbulent situations). Other tests include total petroleum hydrocarbons, which measure the petroleum based fraction of oil and grease. Other sources of oil and grease include animal and vegetable. For BMPs which are designed to address oil and grease, it is suggested that some multiple, within a storm, grab sample analyses would be appropriate. For most BMPs, it is recommended that the

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parameter be optional. If completed, the TPH evaluation is recommended as the most appropriate measure to gauge effectiveness of a BMP at reducing man-induced sources of petroleum oil and greases.

Pesticides/Herbicides. Pesticides and herbicides are regularly detected in urban runoff. However, the number of constituents usually detected is low and most often at levels below available criteria. In Portland, Oregon (WCC, 1993a) the frequency of detection of pesticides herbicides was less than 1% of all the pesticides and herbicides tested. However, the city has noted locations where pesticide concentrations in sediments are high. This could indicate that the problem might be due to misuse or dumping, rather than a general stormwater problem. Although it is possible that pesticides accumulate in sediments from low concentrations in stormwater, some regional assessments of the effectiveness of source control measures (education, identification and elimination of dumping problems) are needed. The Alameda County, CA monitoring program (Cooke and Lee, 1993) and other studies have recently identified that the pesticide Diazinon may be a primary cause of toxicity at very low concentrations (below 8140 method detection limits) to *cerodaphrin dubia* in receiving streams in the south bay area of San Francisco. More research is needed to further define the level of this problem in relation to the actual instream biota, rather than test organisms. At this time, I would not recommend including the pesticide in a standard list, but research studies on the magnitude of the problem and the effectiveness of BMPs on these pesticides should be performed. Due to the low values at which these constituents can cause problems, it would be very difficult to assess BMP performance on a wide-scale basis. For example, it may be more appropriate to eliminate or control the use of Diazinon rather than research BMP effectiveness on concentrations that are below 1 ppb.

Volatile and Semi-Volatile Organics. These pollutants have not generally been detected at a high frequency and in quantities that exceed available criteria [with the exception of Polynuclear Aromatic Hydrocarbons (PAHs), which are discussed separately]. In the recent City of Portland and Eugene sampling programs (WCC, 1993a and 1993b) detection rates were less than 2% of all the tested constituents and below all available criteria. These parameters are not recommended for general analysis unless a BMP effectiveness study is being conducted in an industrial area suspected or known to have elevated levels of organics.

Polynuclear Aromatic Hydrocarbons. The carcinogenic properties of PAHs have generated increased interest in the study of their sources, transport, fate, and aquatic toxicity. Major sources include the combustion of fossil fuels, uncombusted petroleum products (fuels, etc.), and natural and man-caused fires. PAHs have recently been analyzed utilizing detection levels that are significantly below those achieved utilizing the standard semi-volatile organic scans (WCC, 1993a and 1993b; Cooke and Lee, 1993). These tests (GC-MS methods at the nanogram per liter level) have shown that PAHs in stormwater are above human consumption criteria by significant amounts (up to over 100 times). However, these tests are specialized (only a few laboratories provide this level of analysis) and expensive (about $500 to $600 per analysis). In addition, there are no criteria for aquatic life, and toxicity identification evaluations performed in the San Francisco Bay Area have not identified PAHs as the source of toxicity in either developed land-use runoff or in stream stations. For these reasons, PAHs are not recommended for the standard list of constituents to be monitored. However, because of their carcinogenic nature and their tendency to bioaccumulate, new studies may identify potential long-term aquatic life impacts that may require reevaluation of this recommendation.

Data Reporting

Practical and technical data reporting considerations, including consistent formatting of data, the clear indication of QA/QC results, standard comparisons to water quality criteria, reporting of tributary watershed characteristics, and BMP design information would facilitate data usefulness. The last two items are considered critical for evaluation of what contributed to BMP effectiveness in one location over another.

Data Formatting. It is recommended that all constituent concentration data be reported as event mean concentrations (EMCs). Table 2 is an example format for reporting storm event EMCs. It indicates the date of the storm, the EMC value for each sampling point, the data that are estimates based upon QA/QC evaluations, method used for analysis, and detection limit achieved. Also included are summary statistics of the EMCs. These statistics should be based on use of the lognormal distribution. The NURP and FHWA studies (EPA, 1983; Driscoll et al., 1983) identified the lognormal distribution as suitable for characterizing EMC distributions. An example of the variability in data is shown in Figure 5. The figure shows a log-probability plot for total copper collected at a commercial land use station. The event mean concentrations ranged from 6 to 70 µg/l. This high degree of variability is why proper statistical techniques should be employed to evaluate whether a measured difference between BMP before/after or input/output is truly different.

The inclusion of outlet data as a part of any paper or report will allow comparisons of typical outlet concentrations and may allow the determination of the lowest or average expected concentration from a particular type of BMP. For example, it may be that wet ponds may only be able to treat to some minimum concentration range at the outlet and the "effectiveness" is greatly impacted by the inlet concentrations.

Quality Assurance/Quality Control (QA/QC). All monitoring studies should include a QA/QC program. The results of the QA/QC program should be reported in monitoring study reports and summarized in papers. It is especially important to discuss when data are characterized as estimates due to QA/QC results and when detection limits were affected. Too often this information is not included.

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Table 2. Example Data Reporting Table from Eugene NPDES Monitoring Summary Report (Woodward-Clyde Consultants, 1993b)

Chromium	(mg/L)			Method EPA 7191				Receiving Water Quality Criteria**
Storm Event	Date Sample	R-1	C-1	I-1	I-2	M-1	M-2	Detection Limit
#1	9/23/92	0.034	0.016	0.031	0.008		0.003	*0.001*
#2	12/5/92	0.005		0.005	0.003	0.001		*0.001*
#3	12/16/92	0.004	0.004	0.008	0.009		0.003	*0.001*
#4	1/19/93	0.004	0.012	0.019	0.011	0.008	0.004	*0.001*
#5	3/14/93	0.003	0.006	0.020	0.017	0.004	0.007	*0.001*
#6								
#7								
#8								
	Median	0.006	0.008	0.014	0.008	0.003	0.004	
	COV	1.27	0.70	0.86	0.71	–	0.42	
	Mean	0.010	0.010	0.018	0.010	–	0.004	

Copper	(mg/L)			Method EPA 6010				Receiving Water Quality Criteria**
Storm Event	Date Sample	R-1	C-1	I-1	I-2	M-1	M-2	Detection Limit
#1	9/23/92	0.081	0.130	0.071	0.016		0.019	0.001
#2	12/5/92	0.004		0.01	0.01	0.009		0.001
#3	12/16/92	0.011	0.016	0.037	0.03		0.009	0.004
#4	1/19/93	0.009	0.046	0.076	0.034	0.027	0.012	0.003
#5	3/14/93	<0.030	0.027	0.034	0.025	0.020	<0.030	0.004
#6								
#7								
#8								
	Median	0.012	0.040	0.037	0.021	0.017	0.013	
	COV	2.03	1.11	0.97	0.54	–	–	
	Mean	0.030	0.060	0.051	0.024	–	–	

Results expressed as mg/L (ppm) unless otherwise noted. COV is the Coefficient of Variation. ** Criteria are hardness dependent.
"nd" means none detected at or above the detection limit listed. If no value is shown, the lab analysis was not performed.
Summary statistics are based on the assumption that the samples of EMCs are lognormally distributed.
Italicized values are considered estimates due to QA/QC review but are included in the calculations.

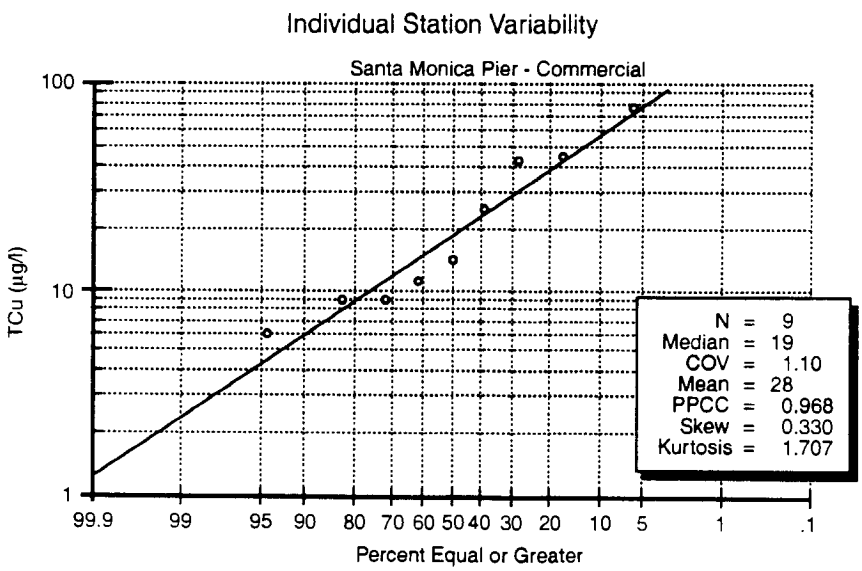

Figure 5. Example log probability plot of storm event mean concentrations from data collected by the Los Angeles County Stormwater Monitoring Program at the Santa Monica Pier (Santa Monica, CA) Commercial Land Use Station.

Comparisons to Water Quality Criteria. Another method to gauge effectiveness could be to monitor how the BMP effects the number of times that criteria are exceeded in both the inflow and the outflow, to assess how the BMP reduces (or does not reduce) the frequency of storm events where water quality criteria are exceeded. For heavy metals analyses, it is recommended that hardness be collected for all storms monitored and that comparisons to criteria be made utilizing the dissolved fraction with the computed aquatic criteria as modified by EPA (1993b). Figure 6 presents an example presentation of metals exceedances for data collected in Portland, OR (WCC 1993a). These data could be compared to BMP data for exceedances to determine whether or not a BMP was actually reducing potential toxicity.

Watershed BMP Design Parameters. Urbonas (1995) described information that should be collected regarding the physical, climatic, and geologic parameters, which include watershed and BMP design characteristics that could likely affect the performance of a BMP. Table 3 (Strecker and Urbonas, 1995) presents a summary of these parameters. More detailed and updated lists will be published upon completion of the EPA study referenced earlier.

Estimation of Pollutant Removal Effectiveness

BMP pollutant removal effectiveness estimations are not straightforward and a wide variety of methods have been employed. Martin and Smoot (1986) discussed the following three types of methods to compute efficiencies:

- The first method employs an efficiency ratio (ER), which is defined in terms of the average event mean concentration (EMC) of pollutants from inflows and outflows, thus:

$$ER = 1 - \frac{\text{Average outlet EMC}}{\text{Average inlet EMC}}$$

- The second method is based on the summation of loads (SOL) of pollutants removed during the monitored storms, thus:

$$SOL = 1 - \frac{\text{Sum of outlet loads}}{\text{Sum of inlet loads}}$$

- The third method of determining efficiency, developed by Martin and Smoot (1986), defines the ratio as the slope of a simple linear regression of inlet loads and outlet loads of pollutants. The equation for the regression of loads (ROL) efficiency is thus:

$$\text{Loads in} = \beta \cdot \text{Loads out}$$

where β equals the slope of the regression line, with the intercept constrained at zero.

The ER and SOL methods assume that monitored storms include samples representative of all storms that occur. The SOL method assumes that enough samples were collected so that any significant input loads or output loads were not missed. They are different in that one gauges effectiveness in terms of concentration reduction, while the other gauges effectiveness in terms of load of pollutant removed. The ROL method assumes that the treatment efficiency is the same for all storms, which is likely not the case.

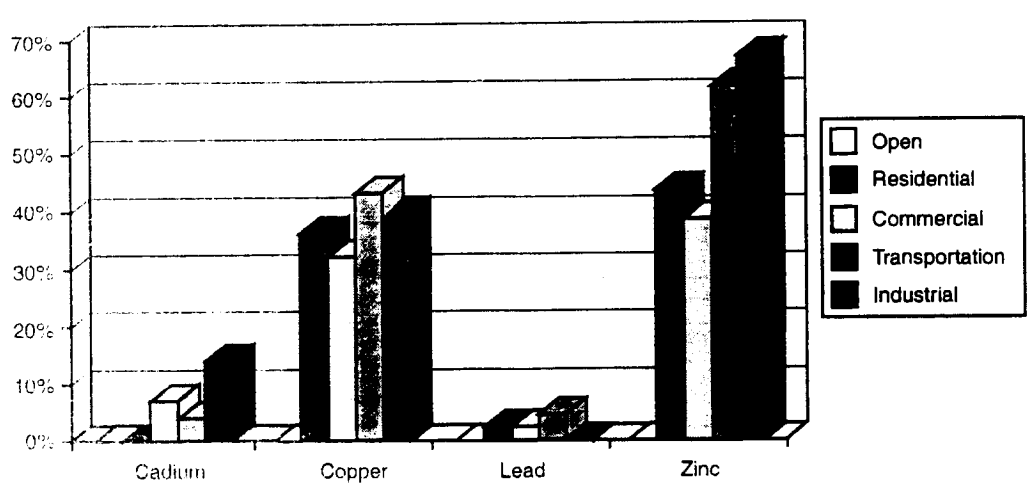

Figure 6. Frequency of water quality criteria exceedances of Oregon urban stormwater data collected for the Municipal Stormwater NPDES Programs.

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Table 3. Parameters to Report with Water Quality Data for Various BMPs

Parameter Type	Parameter (1)	Retention (Wet) Pond (2)	Extended Detention Basin (3)	Wetland Pond Basin (4)	Grass/Swale Wetland Channel (5)	Sand/Leaf Compost Filter (6)	Oil & Sand Trap (Vault) (7)	Infiltration and Percolation (8)
Tributary Watershed	Tributary watershed area	•	•	•	•	•	•	•
	Total tributary watershed impervious percentage	•	•	•	•	•	•	•
	Percent of impervious area hyd. connected	•	•	•	•	•	•	•
	Gutter, sewer, swale, ditches in watershed?	•	•	•	•	•	•	•
	Land use types (res, comm, ind. open) and acreages	•	•	•	•	•	•	•
General Hydrology	Average storm runoff volume	•	•	•	•	•	•	•
	50th percentile storm runoff volume	•	•	•	•	•	•	•
	Coefficient of variation of runoff volumes	•	•	•	•	•	•	•
	Average daily base flow volume	•	•	•	•	•	•	•
	Average runoff interevent time	•	•	•	•	•	•	•
	50th percentile interevent time	•	•	•	•	•	•	•
	Coefficient of variation of interevent times	•	•	•	•	•	•	•
	Average storm duration	•	•	•	•	•	•	•
	50th percentile storm duration	•	•	•	•	•	•	•
	Coefficient of variation of storm durations	•	•	•	•	•	•	•
	2-year flood peak velocity				•		•	
	Depth high groundwater of impermeable layer		•	•				•
Water	Water temperature	•	•	•	•	•	•	•
	Alkalinity, hardness and pH	•	•	•	•	•	•	•
	Sediment setting velocity distribution, when available	•	•	•	•	•	•	•
	Facility on- or off-line?	•	•	•	•	•	•	•
	If off-line, amount of flow bypassed annually	•	•	•	•	•	•	•
General Facility	Type and frequency of maintenance	•	•	•	•	•	•	•
	Inlet and outlet dimensions and details	•	•	•	•	•	•	•
Wet Pool	Solar radiation, when available	•		•	•			
	Volume of permanent pool	•		•		•	•	
	Permanent pool surface area	•		•		•	•	
	Littoral zone surface area	•						
	Length of permanent pool	•		•		•	•	

(continued)

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Table 3. Continued

Parameter Type	Parameter (1)	Retention (Wet) Pond (2)	Extended Detention Basin (3)	Wetland Pond Basin (4)	Grass/Swale Wetland Channel (5)	Sand/Leaf Compost Filter (6)	Oil & Sand Trap (Vault) (7)	Infiltration and Percolation (8)
Detention Volume	Detention (or surcharge) volume	•	•	•		•	•	•
	Detention basin's surface area	•	•	•		•	•	•
	Length of detention basin		•	•		•	•	•
	Brimfull emptying time	•	•	•		•	•	•
	Half-brimfull emptying time	•	•	•		•	•	•
	Bottom stage volume		•					
	Bottom stage surface area		•					
Pre-Treatment	Forebay volume	•	•	•		•	•	•
	Forebay length	•	•	•		•	•	•
	Other BMPs upstream?	•	•	•	•	•	•	•
Wetland Plant	Wetland type, rock filter present?			•	•			
	Percent of wetland surface at $P_{0.3}$ and $P_{0.6}$ depths			•	•			
	Meadow wetland surface area			•	•			
	Plant species and age of facility	•	•	•	•			

Adapted from Urbonas (1995)

Some researchers have suggested that one should utilize an efficiency measure based upon storm pollutant loads into and out of the BMP on a storm-by-storm basis. This would weight the effectiveness considering that all storms are "equal" in computing the average removal. However, it is readily apparent that all storm volumes and their associated concentrations are not equal. Similarly one could utilize concentrations on a storm-by-storm basis.

One factor that complicates the estimation of effectiveness is that, for wet ponds and wetlands (and other BMPs where there is a permanent pool), comparing effectiveness on a storm-by-storm basis neglects the fact that the outflow being measured may have a limited or no relationship to the inflow. In analysis of rain gauges utilizing SYNOP (Driscoll, et al., 1989), if a basin sized to have a permanent pool equal to the average storm, about 60 to 70% of the storms would be less than this volume. In many cases, the flows leaving may have little or no contribution to flows entering the pond. Therefore, storm-to-storm comparisons are probably not valid. In cases like this, it is probably more appropriate to utilize statistical characterizations of the inflow and outflow concentrations to evaluate effectiveness or, if enough samples are collected (i.e., almost all storms monitored), to utilize total loads into and out of the BMP.

Using the same set of data, Table 4 compares three of the methods including percent removal by storm with a statistical characterization of inflow/outflow concentration and a simple comparison of total loads in and out for the sampled storms. As one can see, the removals estimated differ by up to 19 percentage points. In this record, there are several storm events where inflow concentrations were relatively low and therefore the system was not "effective".

Based upon these factors, it is recommended that the statistical characterization of inflows vs. outflows be utilized (ER). This enhances the ability to conduct statistical tests of whether the reported differences are greater than zero. If enough data on storms are collected (e.g. continuous samples over an extended period), the total loads in and out (SOL) is probably an acceptable method also.

BMP Evaluations – Statistical Considerations

As noted in many studies of urban runoff, the variability in runoff concentrations from event to event is large. If one were to attempt to statistically characterize a BMP influent concentration (and outflow), the more data the better, Figure 7 is a schematic of how more data can improve (reduce confidence interval of) results. As mentioned above, there are a number of types of BMP evaluations that can be conducted. First, the standard evaluation of a single BMP, testing input and output; second, the evaluation of multiple BMPs within a basin (before/after or control ba-

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Table 4. Example Wetland TSS Removal

Storm	Volume of (ft³) Inflow = Outflow	Concentration In (mg/L)	Concentration Out (mg/L)	Load In (lbs)	Load Out (lbs)		% Removal by storm
1	445,300	352	24	9780	670		93%
2	649,800	30	25	1220	1010		17
3	456,100	99	83	2820	2360		16
4	348,111	433	141	9410	3060		67
5	730,261	115	63	5240	2870		45
	Med	139	65			A	
	Cov	1.48	.86	28,470	9,970	V	48%
	Mean	249	85			G	
		Conc	66%	Loads	65%		

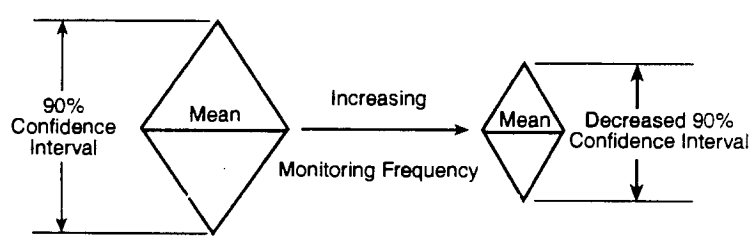

Figure 7. Expected change of 90% confidence interval of station mean with additional data.

sin); and finally a third, the evaluation of a BMP with multiple inlets (where it might be very difficult (expensive) to evaluate the BMP utilizing input/output). All methods should require that a rigorous statistical approach be applied in selecting the number of samples to be collected to assure detection of a given level of change.

As an example of the number of samples required to detect a "true" difference, Table 5 presents an analysis of two of the Portland NPDES monitoring stations (WCC, 1993a) where 10 flow-weighted composited samples were collected. The Fanno Creek station is a large (about 1,200 acres) residential catchment, while the M1 station is a smaller (about 100 acres) mixed land use station. An analysis of a variance-based test was utilized with the existing data to determine how many samples are estimated to be needed to detect a 5%, 20%, and 50% change in the mean concentration at the station. The test was performed considering an 80% probability that the difference will be found to be significant, with a 5% level of significance (Sokal and Rohlf, 1969). This analysis does not consider potential seasonal effects on the collection of data as a factor. Even so, quite a large number of samples would be required to detect a 5% to 20% difference in concentrations. Figure 8 shows a map of the US plotting the average number of storms per year (over 0.1") as determined by EPA (Driscoll et al., 1989) occur. One can see that in many locations, it would take a number of years of sampling all storm events to be able to detect small differences.

There are numerous examples in the literature where small differences (2 to 5%) are reported based upon much fewer samples than indicated by this analysis. This highlights the need to be more rigorous with regard to statistical testing of reported effectiveness estimates. To detect larger changes, the number of samples becomes reasonable. The mixed land use catchment in Portland is currently being studied for the effectiveness of the implementation of a number of source controls and other controls that do not lend themselves to input/output testing. Examples include maintenance changes (catch basin cleaning, street sweeping), education (business and residences), tree planting, etc. Post-BMP monitoring will be conducted along with qualitative evaluations.

As an example that demonstrates how one could evaluate whether one catchment is different than another, Figure 9 presents results of analysis of stormwater monitoring data collected in Oregon. The figure presents a statistical characterization of land use data, demonstrating that for Total Copper, the open and residential land use stations are statistically different from all other land uses as well as from each other. A similar analysis technique should be employed for all before and after tests, as well as "control" tests.

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Table 5. Analysis of Sample Sizes Needed to Statistically Detect Changes in Mean Pollutant Concentrations from 2 Stations in Portland, OR.

		Number of Samples Required to Detect the Indicated % Reduction in Site Mean Concentration*		
Monitoring Site	Parameter	5%	20%	50%
R1 - Fanno Creek Residential	TSS	202	14	4
	Copper	442	29	6
	Phosphorus	244	16	4
M1 - NE 122nd Columbia Slough Mixed Use	TSS	61	5	2
	Copper	226	15	4
	Phosphorus	105	8	3

*80% certain of detecting the indicated % reduction in mean of the EMCs.

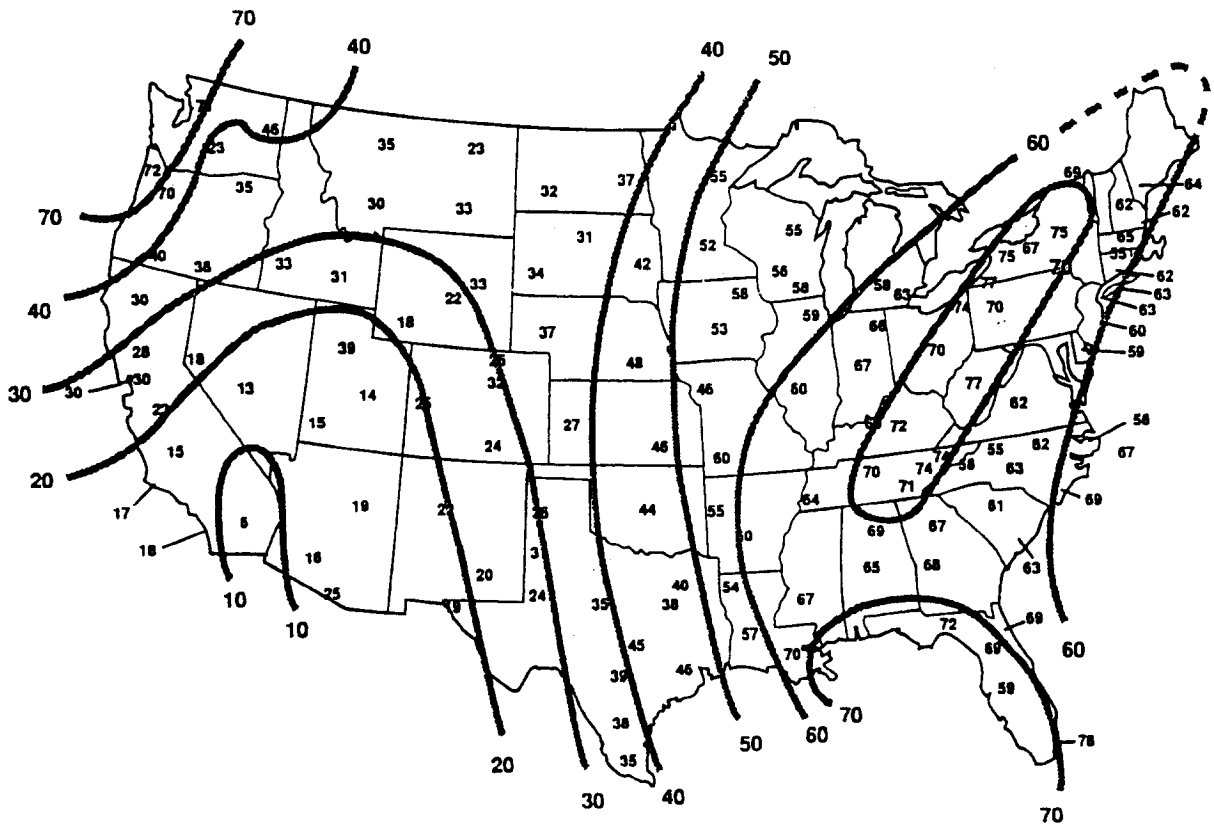

Figure 8. Annual average number of storms. (storms/year)

Other Considerations

There is a need to conduct dry weather analyses between storms on BMPs with dry weather flows; it may be that pollutants captured during storms are slowly released during dry weather discharges.

Biological and downstream physical habitat assessments such as aquatic invertebrate sampling and habitat classification should be explored as an alternative to merely utilizing chemical measures of effectiveness (see Maxted, these proceedings); long-term trends in receiving water quality, coupled with biological assessments, would likely be a much better gauge of the success of the implementation of BMPs, especially on an area-wide basis.

Summary and Recommendations

There is a great need for consistency in the constituents and methods utilized for assessing BMP effectiveness. This

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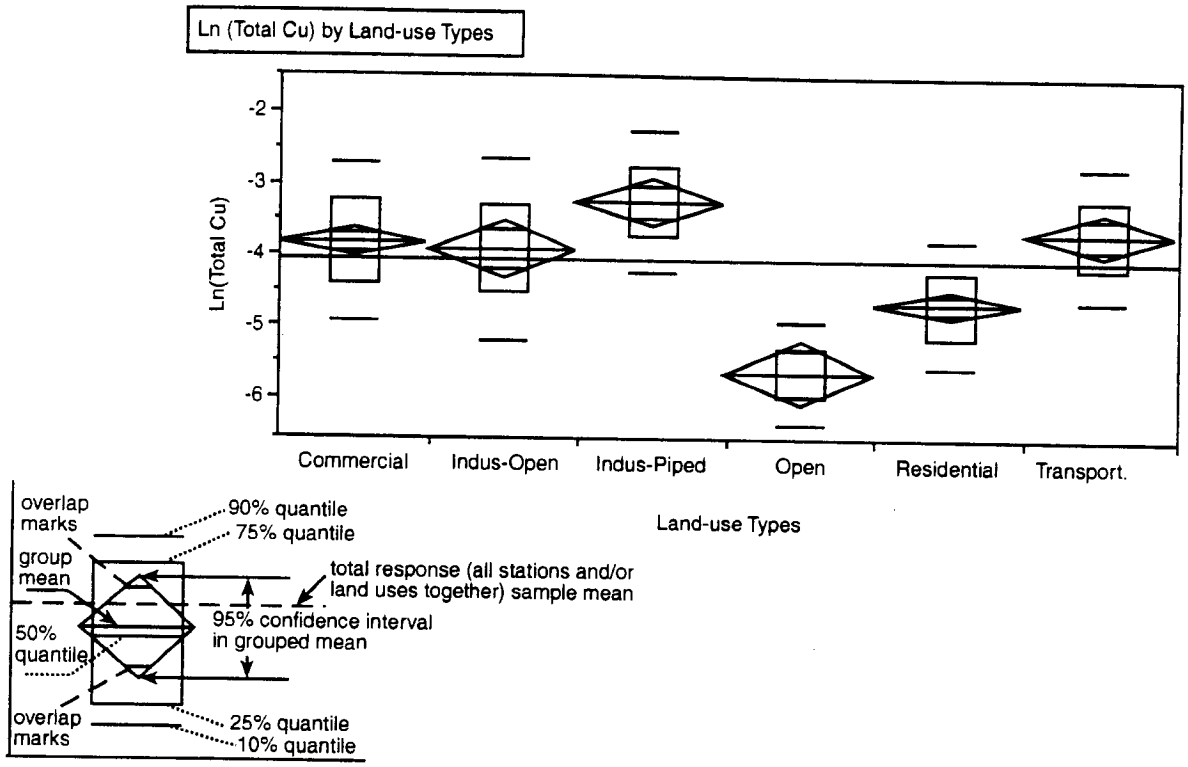

Figure 9. Box Plots of land-use event mean concentration for Oregon stormwater collected for the Municipal Stormwater NPDES Program. (Strecker, et al., 1997)

paper has presented only some of the consistency issues. It is recommended that researchers who undertake BMP effectiveness studies consider the recommendations suggested here, and by Urbonas (1995). It is the authors' opinion that EPA should require studies receiving federal funding to conduct BMP effectiveness studies which utilize standard methods (as suggested here) together with (still much needed) detailed guidance on data collection and sampling methods to improve data transferability.

Acknowledgments

The author wishes to thank Wayne Huber, Gene Driscoll, Gail Boyd, Ben Urbonas, Jonathan Jones, Tom Schueler, Marcus Quigley, and Krista Reininga for their helpful discussions and comments on the subject.

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Targets of Opportunity: Alexandria's Urban Retrofit Program

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During 1992 preparations for a stormwater quality program as part of the Alexandria (Virginia) Chesapeake Bay Preservation Ordinance, the city engineering staff made a survey to identify opportunities for future urban BMP retrofitting. The objective of the current "Targets of Opportunity" program is to enhance the minimum requirements of the Virginia Chesapeake Bay Preservation Act (CBPA) program by providing treatment of stormwater runoff from built-up areas not directly addressed by the CBPA, in order to further reduce pollutants reaching the bay and its tributaries.

Since the inception of the Targets of Opportunity Program, almost 1,000 acres of urban BMP retrofits have been installed within Alexandria. A substantial part of Alexandria's urban retrofits have been voluntarily designed and constructed by developers of adjacent downhill properties. While comprising only 3.3% of the urbanized area within the Potomac and Shenandoah basin, the city has already contributed almost 23% of the total urban retrofit coverage proposed in the *Shenandoah and Potomac River Basins Tributary Nutrient Reduction Strategy* (Commonwealth of Virginia, 1996.) Estimated nutrient reductions already exceed the urban retrofit phosphorus and nitrogen reduction targets contained in the Shenandoah Strategy, and are within 3% of meeting the nitrogen reduction target.

Characteristics of Alexandria

Alexandria is a city of approximately 115,000 citizens located on the Potomac River in Northern Virginia between Fairfax County and Arlington County. Founded in 1749 as a deep river seaport, the city is currently some 15.75 square miles in size. The population is diverse and relatively affluent, some 56,000 households with a median income of approximately $53,000. Directly across the Potomac from Washington, D.C., Alexandria's largest employer remains the federal government. However, extensive development during the past decade has resulted in a thriving commercial sector, with divisional, regional, and multinational headquarters for operations ranging from research and development to high technology, associations, and professional services now located in the city. With a 1990 population density of 7,281 people per square mile. Alexandria is the most densely developed city in Virginia and the eleventh most densely populated city in the United States. Approximately 41% of the total city area is covered with impervious surfaces.

Alexandria is a city bounded by and laced with streams. The eastern boundary is 5.6 miles of Potomac River shoreline. The northern boundary with Arlington County includes 1.9 miles of Four Mile Run, 100-year flood channel reconstructed by the U.S. Army Corps of Engineers in the 1970s following extensive flooding during Hurricane Agnes. A similar 1.7 miles of channel conveying Cameron Run borders the Capital Beltway, the southern border with Fairfax County. Approximately 8.4 miles of small tributary streams flow north or south into the boundary channels, approximately 20% to Four Mile Run, 20% directly into the Potomac River, and 60% into Cameron Run or its major tributaries, Backlick Run and Holmes Run. Almost all of Alexandria's streams except the Potomac River are severely degraded urban streams. Protection and partial restoration of these streams and the Chesapeake Bay into which they flow is the focus of the city's nonpoint source programs.

Governing Clean Water Programs

Alexandria's clean water programs are governed by several federal and state authorities. Initially, the city was classified as a medium-sized city under the National Pollution Discharge Elimination System (NPDES) Permit requirements for separate stormwater systems, requiring an MS4 permit. (The city was later reclassified as a small city based on population reductions allowed for regions served by permitted combined sewer systems). Virginia has also adopted a number of programs in support of the federal and multi-state Chesapeake Bay Program. The Virginia CBPA includes a mandatory program requiring provisions for stormwater quality on development projects within the bay watershed.

The Virginia Stormwater Management Act is a discretionary program which allows stormwater quality to be required of development projects throughout the commonwealth. The Virginia Erosion and Sediment Control Act requires stormwater quality measures during construction

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on all but very small development projects. The Chesapeake Bay Tributary Nutrient Strategies require nutrient reductions from all point and nonpoint sources to achieve 40% reductions in the Bay. These reductions to get nutrient levels below the 1985 baseline have been targeted for the year 2000 by the bay program signatories. Only the Tributary Strategies address the question of urban BMP (Best Management Practices) retrofits.

Tributary Targets

The Shenandoah and Potomac River Basins Tributary Nutrient Reduction Strategy (the Strategy) defines urban BMP retrofits as "Modifying existing stormwater facilities to enhance water quality and/or retrofitting stormwater drainage systems to add water quality components in already developed areas to slow runoff, remove sediment and nutrients, and provide a basis for restoring eroded stream channels." The strategy sets a target of 4,356 acres of urban retrofit within the entire basin, of which 1,156 acres was in addition to that existing at the time of the printing. The Strategy lists total urbanized watershed as 454 square miles (290,400 acres), including Alexandria's 15.75 square miles (10,080 acres), approximately 3.5% of the watershed.

Targets of Opportunity Program

Stormwater Quality Program Adoption

Alexandria's initial stormwater quality program was its erosion and sediment control ordinance, which was adopted in the 1970s. In 1990, the city began an intensive effort to prepare an application for an NPDES permit for its stormwater program. Concurrently, city staff enacted a Chesapeake Bay Preservation Ordinance (City of Alexandria, 1991) which contains provisions from both the Chesapeake Bay Preservation Act and the Stormwater Management Act. City staff identified several sites where urban retrofits appeared possible. Staff members responsible for reviewing development proposals were directed to discuss the possibility of including urban retrofit as part of proposed developments. The objective of this program was to enhance the mandatory requirements of the Chesapeake Bay Preservation program with additional treatment of stormwater runoff from built-up areas not directly addressed by that act, to further reduce pollutants reaching the bay and its tributaries. The program was already in place and functioning when the Virginia Potomac Basin Tributary Nutrient Reduction Strategy was developed and adopted in 1995-1996 (Commonwealth of Virginia, 1996).

Elements of the Targets of Opportunity Program

There are four basic elements to the Targets of Opportunity Program. The first is knowledge of the watersheds within the jurisdiction. The Alexandria staff used aerial photographs, topographic maps, and the sewer outfall map prepared for the Part I NPDES Stormwater submission. Discussions with storm and sanitary sewer maintenance personnel with many years of experience in the city were also very valuable.

The second element is the identification of potential opportunities for urban retrofits. Sites in the watersheds of streams which receive large numbers of stormwater outfalls are especially desirable to maximize the effectiveness of retrofit BMPs. Alexandria also focused on areas with existing ponds and detention basins which could be adapted in the future for service as either regional retention basins (wet ponds) or extended detention basins (dry ponds).

The third element is one of the most crucial: early exploration of urban retrofit options with owners/developers. Alexandria's zoning ordinance requires a pre-submission conference with the city staff for all significant construction projects. This conference usually occurs prior to finalization of the stormwater concept plan for the respective site, allowing the staff an opportunity to discuss retrofit options with the development team. For smaller projects, the staff almost always becomes aware of proposed development well before formal submission through informal contacts with the engineering community.

Once contact is established, the fourth element comes into play: creating "win-win" situations for both the developers and the public. In some cases, developers may find it less expensive to treat the entire flow of existing storm sewers transiting a site rather than construct a separate "off-line" system to collect and treat stormwater runoff. Alternatively, construction (at the developer's expense) of regional facilities on public land may be more economically beneficial to a developer than construction on-site BMPs. Fostering a spirit of cooperation between the parties rather than an adversarial regulator/regulated relationship is crucial to obtaining results in a program of this nature.

Status of the Targets of Opportunity Program

Seven significant urban stormwater BMP retrofit projects have been approved under the Targets of Opportunity Program. A discussion of each of the projects illustrates how the various program elements were implemented.

Winkler Run Regional Retention Facility

The Mark Winkler Corporation, which owns a large development tract in western Alexandria, proposed to construct a combined stormwater detention/water quality pond system to stop a severe erosion problem and to provide detention and water quality for future buildout of their property. Noting that the watershed draining through the site included significant built-up areas, the staff discussed with the developer's engineer the possibility of sizing the pond to provide water quality for the entire watershed at full buildout under city zoning. The developer agreed and of the total watershed of 221 acres, 126.7 acres was urban retrofit. Estimated impervious cover on this acreage, which includes 26 acres of Interstate 395 and the heavily traveled Seminary Road and Beauregard Street, apartment houses, and hotel and office complexes, is 82.3%. The Winkler Run Pond System was built to a design from the

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Washington Council of Governments manual, *Controlling Urban Runoff*, (Schueler, 1987) and is rated to provide 45% phosphorous removal. The system of two ponds in series was constructed in 1992. Based on information in the manual, city staff estimate total nitrogen removal at approximately 30% (see the Attachment for nutrient estimating methodology). At these ratings, the total yearly reductions from the urban retrofit areas are 397 pounds per year of phosphorus and 1,960 pounds per year of nitrogen. The system was constructed solely at the developer's expense.

Lake Cook Regional Retention Facility

The next significant urban retrofit involved a project to restore a viable habitat for aquatic life in Lake Cook, a recreational lake owned by the city and operated by the Northern Virginia Regional Park Authority (NVRPA). In 1993, NVRPA requested the city to restore sufficient depth to maintain for recreational fish in Lake Cook in the Cameron Run Regional Park. Originally four feet deep, the three-acre lake had silted until less than two feet of depth remained. During an earlier review of the outfall map, the engineering staff had noted that the lake was fed by Strawberry Run, a stream receiving the outfalls from over 30 storm sewers having a diameter of 36 inches or greater. The staff determined that if the lake were deepened to an average of six feet, the pool could serve as a regional wet pond BMP for approximately the 385 acres of fully developed watershed draining into the lake. A sediment forebay was also added to trap sediments at the upstream end of the lake, protecting the fish habitat and easing future maintenance.

The lake was deepened during the winter of 1993-1994. Only approximately 20% of the total project cost of $75,000 (funded by Alexandria general revenues) was attributable to the stormwater quality features. Based on the BMP phosphorus removal ratings currently proposed for inclusion within the Virginia Stormwater Management Regulations, Virginia Chesapeake Bay Protection Regulations, and the WASHCOG nitrogen removal estimates for such ponds, the staff estimates that the lake now removes approximately 926 pounds of phosphorus and 4,222 pounds of nitrogen per year from stormwater runoff entering Cameron Run.

Cameron Lake Regional Retention Facility

In the early 1990s, the Department of Defense decided to close the 164.5-acre Cameron Station Army Base in Alexandria and sell the land for private development. While developing future zoning of the property, city staff noted that two connected lakes on the station which would become a city park acted as stormwater detention ponds for approximately 60 acres of runoff. Total surface area of the lakes was almost 5 acres, and their permanent pools could be deepened to serve as a regional retention pond for a much larger area, including some of the most intensely developed areas of the city. Due to early coordination with the Army, the city was able to insert a condition that required all future development on the site to drain through the lakes. The condition also required city storm sewers which transited the base to be rerouted through the lakes. The purchaser of the base readily participated in this public-private partnership, recognizing that it would provide a win-win relationship.

By allowing the developer to retrofit the existing lakes, greater densities of development were created, which more than compensated for the cost of upgrading the retention facility. Additionally, if the developer had held the density of development as originally planned, a series of sand filters would have been needed to provide water quality for the 97.5 acres of development at a cost considerably higher than the retrofitting of the existing lakes. Recognizing that the regional facility would provide a win-win situation, the developer agreed to the incorporation of several state-of-the-art features into the retrofit.

Work began with draining the lake and removing approximately 20,000 cubic yards of material to create an appropriate permanent pool. The existing outlet structure, consisting of little more than a concrete flume with a wire trash rack, was removed and replaced with an upflow anaerobic trickling filter. Additional features include a sediment forebay which can be isolated from the permanent pool during maintenance and an oil skimmer to retain floating hydrocarbons, trash, etc., from reaching the main basin of the lake. Facilities to monitor flow rates through the pond and chemical composition of the flows were also provided. Constructed during the summer of 1997, the regional retention facility is treating runoff from 246.83 acres, 187 acres of which did not previously drain through the lakes. The entire drainage shed except for the 97.5 acres of development property is urban retrofit. Based on Virginia and WASHCOG BMP ratings, the staff estimates that, when full buildout of Cameron Station is completed, the facility will remove approximately 709 pounds of phosphorus per year and 3,235 pounds of nitrogen from stormwater runoff entering Backlick Run.

Park Center Regional Extended Detention Facility

Since 1992, the city staff has been recommending the conversion of a large 100-year storm detention facility in the western part of the city into a stormwater quality BMP. Early plans to convert it to a regional retention facility collapsed when one of the adjacent property owners objected to the presence of a permanent pool. However, in the fall of 1996, the owner of the basin submitted a development plan to construct new office towers adjacent to the basin. Rather than require construction of new BMPs for the 8.12-acre development, the city proposed to the developer's engineer that the basin be converted into a regional extended detention facility to serve a 245-acre watershed. An adequate dam and riser structure able to accommodate a 100-year storm was already in place; the conversion involved only modifications to the riser structure to provide a reduced orifice at the bottom of the dam and new overflow openings at the top of the new BMP detention pool. The entire expense of this construction, which is currently in progress, is being borne by the developer. Using the Virginia BMP rating of 35% for phosphorus reduction,

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the staff estimates that the 237 acres of urban retrofit is removing approximately 307 pounds of phosphorous from Lucky Run, a tributary of Four-Mile Run and the Potomac River. The staff is not currently convinced that significant nitrogen removal occurs in extended detention ponds.

Slater's Village Regional Extended Detention Facility

When a development with 145 townhouses and 128 condominium units was proposed in 1996 for part of the old Potomac Rail Yard at the northern portion of Alexandria, the developer's engineer observed that reinforced concrete storm sewer serving existing developments traversed the site. It was determined that it would be less expensive for the developer to construct an extended detention facility to treat all of the runoff conveyed by this sewer than to construct a completely separate storm sewer system and BMP to serve only the new townhouses. He therefore proposed to drain the new development directly into the storm sewer and treat the total runoff downstream of the townhouses. This provided an additional 38 acres of pure urban retrofit to the new extended detention facility. Using Virginia BMP ratings, the staff estimates that this urban retrofit removes an additional 75 pounds of phosphorus from runoff flowing directly into the Potomac River. The BMP was constructed as an erosion control basin in the summer of 1997 and will be converted to a full BMP upon completion of the construction project.

Episcopal Seminary Regional Retention Facility

The Episcopal Theological Seminary, a large landholder in central Alexandria, recently decided to construct a state-of-the-art stormwater retention facility to use as a teaching tool for environmental classes at the private high school on site. The pond, was also designed as a stormwater retrofit pond to serve existing development on the property to provide stormwater quality for any future expansion of facilities. The pond serves a 51-acre watershed with an ultimate runoff factor of 0.44. The city staff considers this private pond to be totally urban retrofit. Initial estimates suggest that this BMP will remove 128.5 pounds per year of phosphorus and 586 pounds per year of nitrogen currently reaching Cameron Run and the Potomac River.

Potomac Retail Center Urban Retrofit

When design began on a 60-acre shopping center to occupy part of a former rail yard in the northern part of Alexandria, the developer's engineer was required to deal with the runoff from 9.9 acres near U.S. Route 1 including adjacent properties which already drained through ditches in the rail yard. Rather than provide a separate conveyance for this off-site water, the engineer proposed to route it through a large retention pond being built to treat development runoff. The retention pond, which included up-to-date features such as a sediment forebay, was sized accordingly. The city staff estimates that this urban retrofit is removing approximately 31.8 pounds/year of phosphorus and 188.7 pounds per year of nitrogen.

Program Impact on the Chesapeake Bay

The seven completed urban BMP retrofit projects described above have provided a total of 996.8 acres of urban retrofit since 1992, 23% of the Potomac/Shenandoah basin total target and 82.9% of the increased coverage target. Total phosphorus removal from these projects is estimated at 2,544.8 pounds per year and total annual removal of nitrogen is estimated at 10,193.0 pounds. The annual phosphorus removal represents 220% of the Nutrient Strategy total basin target and 839% of the increased coverage target. The annual nitrogen reductions represent 97% of the total basin target and 368% of the increased coverage target.

In December of 1997, the City of Alexandria was awarded a Community Innovation Award by the Chesapeake Bay Local Government Advisory Committee for "its contribution and commitment to the protection and restoration of streams, rivers, and the Chesapeake Bay through the implementation of its Stormwater Urban BMP Retrofit Program -- Targets of Opportunity."

Transferability of Program

Any jurisdiction having a formal stormwater quality program, such as the Virginia Stormwater Management Program or Chesapeake Bay Preservation Program, could institute an urban retrofit program similar to Alexandria's. Detailed engineering studies need not be made to begin a program, nor are sophisticated tools such as GIS systems a necessity. A review of aerial photographs of the jurisdiction by staff engineers and storm sewer personnel is usually sufficient to identify the "targets of opportunity" for future urban retrofit upon development or redevelopment in the watersheds.

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Attachment 1
Nutrient Removal Estimates Methodology

The method to calculate loadings recommended by the Virginia Chesapeake Bay Local Assistance Department (CBLAD) and adopted by Alexandria is the Simple Method derived by Thomas R. Schueler in the Metropolitan Washington Council of Governments (COG) handbook, "*Controlling Urban Runoff.*" The Simple Method is described as follows:

$$L = P \times P_i \times R_v \times C \times A \times 2.72/12$$

where,

- L = phosphorus loadings (pounds/year--lb/yr).
- P = average annual rainfall depth (inches) = 40 inches per year for Alexandria.
- P_i = unitless correction factor for storms that produce no runoff = 0.9.
- R_v = runoff coefficient = expresses the fraction of rainfall converted to runoff.
- C = flow-weighted mean pollutant concentration (milligrams/liter--mg/l).
- A = area of development site (acres).
- 2.72 and 12 are conversion constants.

Further reducing the Alexandria constants in the formula yields:

$$L = 8.16 \times R_v \times C \times A$$

The runoff coefficient describes the fraction of rainfall converted to runoff. While dependent on soil type, topography and cover, it is most influenced by watershed imperviousness. The Simple Method uses the following formula to compute R_v:

$$R_v = 0.05 + 0.009\ (I)$$

where

I = the % of site imperviousness in whole numbers.

For watersheds with greater than 20% impervious cover, CBLAD recommends using a flow-weighted mean concentrations of phosphorus of 1.08 mg/l.

A six-month BMP monitoring project in Alexandria in 1994 established an actual flow-weighted mean concentration of nitrogen in stormwater runoff of 8.0 mg/l.

The new Virginia Stormwater Management Regulations (also to be used by CBLAD) rating for retention ponds with 2.0 inches of runoff from impervious surfaces in the permanent pool is 65% for TP.

Based on various studies reviewed, including WASHCOG's *Controlling Urban Runoff*, city staff estimates TN removal for such ponds at 40%. Pending further analysis of monitoring studies, city staff is not currently asserting any TN removal from extended detention facilities.

Retention ponds with permanent pools of less than 2.0 inches of runoff were rated using data from the *Northern Virginia BMP Handbook* and *Controlling Urban Runoff.*

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Attachment 2

Total Phosphorus Reduction Calculations

Project	Urban Retrofit Acres	% Imper-vious	Runoff Factor R_v	Alexandria Constant	C TP (mg/l)	Annual LOAD TP-lb.)	BMP Type	BMP Effic'y (%)	Annual Load Reduced (lbs)
Winkler Run Pond	126.7	82.3	0.79	8.16	1.08	882.1	Ret. Pond	45	397.0
Lake Cook Retrofit	385.0	41.0	0.42	8.16	1.08	1425.0	Ret. Pond	65	926.3
Cameron Lakes Retrofit	149.3	86.6	0.83	8.16	1.08	1092.1	Ret. Pond	65	709.8
Park Center Basin Retrofit	236.9	41.0	0.42	8.16	1.08	876.9	ED Pond	35	306.9
Episcopal Center Pond	51.0	43.3	0.44	8.16	1.08	197.8	Ret. Pond	50	98.9
Slater's Village ED Retrofit	38.0	65.3	0.64	8.16	1.08	197.8	ED Pond	35	75.0
Potomac Yard Shopping Center	9.9	75.5	0.73	8.16	1.08	63.7	Ret. Pond	50	31.8
Total	996.8	–	–	–	–	–	–	–	2554.8

Total Nitrogen Reduction Calculations

Project	Urban Retrofit Acres	% Imper-vious	Runoff Factor R_v	Alexandria Constant	C TN (mg/l)	Annual LOAD TP-lb.)	BMP Type	BMP Effic'y (%)	Annual Load Reduced (lbs)
Winkler Run Pond	126.7	82.3	0.79	8.16	8.0	6534.1	Ret. Pond	30	1960.2
Lake Cook Retrofit	385.0	41.0	0.42	8.16	8.0	10,558.8	Ret. Pond	40	4222.3
Cameron Lakes Retrofit	149.3	86.6	0.83	8.16	8.0	8089.4	Ret. Pond	40	3235.8
Park Center Basin Retrofit	236.9	41.0	0.42	8.16	8.0	9897.5	ED Pond	–	–
Episcopal Center Pond	51.0	43.3	0.44	8.16	8.0	1464.9	Ret. Pond	40	586.0
Slater's Village ED Retrofit	38.0	65.3	0.64	8.16	8.0	1587.6	ED Pond	–	–
Potomac Yard Shopping Center	9.9	75.5	0.73	8.16	8.0	471.8	Ret. Pond	40	188.7
Total	996.8	–	–	–	–	–	–	–	10,193.0

Port Towns Revitalization and Environmental Enhancement - Stormwater Projects Revitalize Urban Areas

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Background

In 1993, Prince Georges County, Maryland (Figure 1) embarked upon an ambitious neighborhood revitalization program that targets communities inside the National Capital Beltway. The primary purpose of this effort is to revive older communities as attractive places to live and work. By concentrating resources and developing local institutions, state and county officials hope to stem disinvestment and abandonment of these communities.

Prince Georges County has a population of 800,000 and a median household income of $60,540. It covers 488 square miles and inclues 28 incorporated areas. Its economic wealth is tied to both Washington, DC, and Baltimore. Business is largely clustered in the northern part of the County. The county's southern area is still largely rural.

The county is bisected, geographically and economically, by the National Capital Beltway. Although the majority of the population is located in the municipalities inside the Beltway, higher income levels are concentrated outside the Beltway (Tatar, et al., 1995).

The centerpiece of the County's revitalization effort is the renewal of communities known as the Port Towns, located in the heart of Prince Georges County. What started as a thriving tobacco port and trading point along the Anacostia River in Colonial days is now a predominantly blue-collar neighborhood. Built mostly in the 1940s and 1950s, the Port Towns have fallen into disrepair and have struggled to attract new residents and businesses (Pierre, 1997).

The Port Towns revitalization initiative has generated public interest and support due to this community's keen sense of identity and heritage. With millions of dollars already earmarked for various projects, the Port Towns community is now one of the most prominent revitalization areas in Maryland.

Stormwater Revitalization Projects

What is unique about the Port Towns is that innovative stormwater retrofit projects are helping to pave the way to economic renewal. Construction of bioretention[1] streetscaping, shallow marsh wetlands, stream rehabilitation, and river restoration projects are being used to revitalize the towns. As a new urban revitalization tool, these stormwater projects are intended to mitigate adverse environmental impacts in older urban areas, improve landscaping, enhance community pride, and create a wholesome community image that invites private investment.

Port Towns

The Towns of Bladensburg, Colmar Manor, and Cottage City are collectively known as the Anacostia "Port Towns" (Figure 2) industrial and commercial activity. The Port Towns residential areas include a wide range of generally pleasant housing along with industrial and commercial activity. Economic decline in the Port Towns began due to relocation of retail shops to newer outlying malls, as well as constraints imposed by older infrastructure.

Facing similar development issues, common economies, and proximity to US Route 1, the three towns agreed to coordinate their revitalization efforts. To guide the revitalization effort, the Port Towns developed a comprehensive Vision and Action Plan in 1995. Although an extensive park or "greenway" along the banks of the Anacostia River makes an important contribution to the character of this community, the Port Towns generally lack adequate trees, streetscaping, and stormwater management controls (Legg Mason et al., 1995).

A New Revitalization Tool - Stormwater Retrofit Projects

For years, the County has considered the restoration of the Anacostia River an important part of its capital program. Although the County viewed the stormwater retrofit of the Anacostia watershed as an environmental goal, the Port Towns saw it as an opportunity to serve an economic

[1] Bioretention BMPs are stormwater retention facilities designed to mimic forested systems that naturally control hydrology through infiltration and evapotranspiration (Prince George's County Low Impact Development Manual, 1997).

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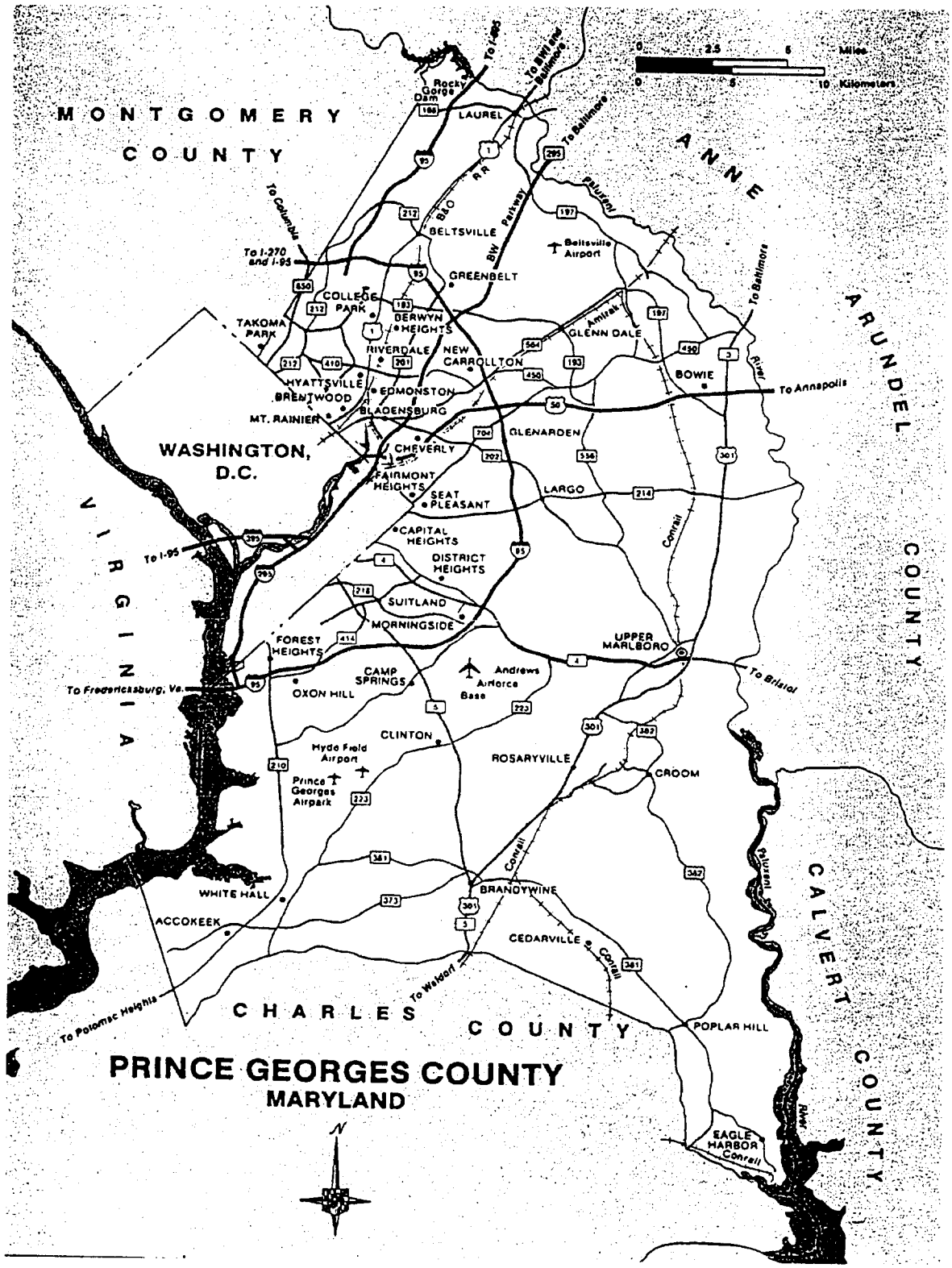

Figure 1. Prince Georges County, MD (white area on map).

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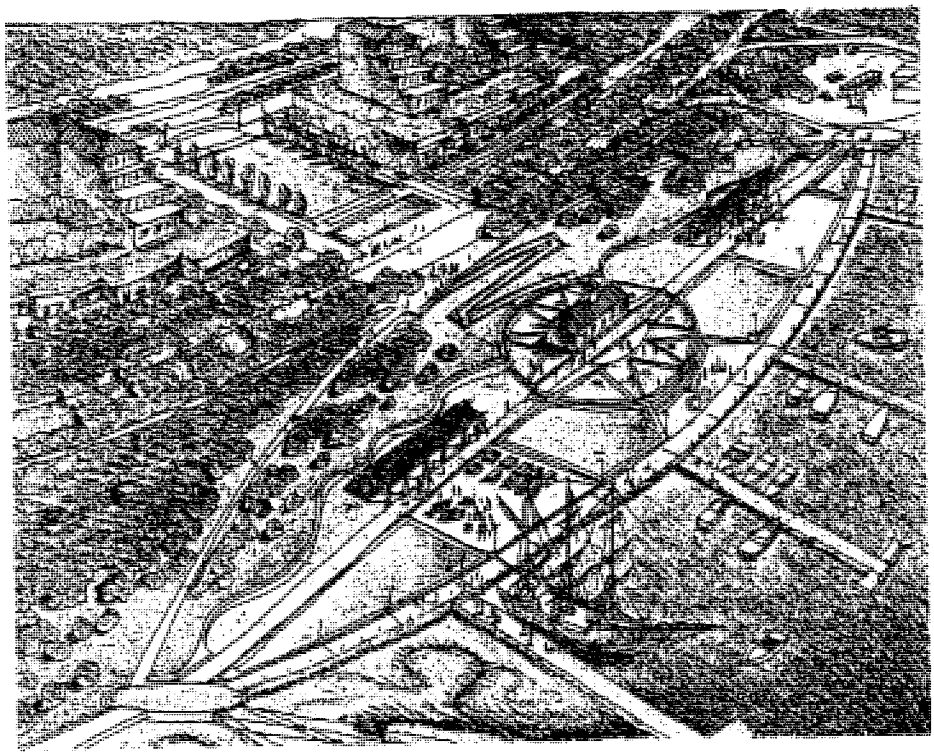

Figure 3. Drawing of the proposed historic Bladensburg Waterfront Park.

Figure 4. Dry ponds converted to shallow marsh systems improve water quality and enhance the aesthetic beauty of the Port Towns community.

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Figure 5. Bioretention streetscaping (planted areas to the right of the street in the photo) help to treat stormwater runoff and improve the appearance of the community.

Table 2. Project Sponsors

- Prince Georges County Government
- Maryland National Capital Parks and Planning Commission
- Port Town Community Development Corporation
- Town governments
- Maryland Department of the Environment
- Maryland Department of Transportation
- US Army Corps of Engineers
- Commercial Management District
- Community

natural resource areas that are important renewal areas. Public exposure and acceptance of the initial pilot projects are important to enhancing visual and spatial impacts. The availability of land and easements is normally confirmed before starting the design phase. To lower project costs and to help knit a public-private partnership, publicly owned or "gratis" private easements are sought first. Permits for bioretention streetscape projects are generally easily acquired. Permits for reconstruction of wetlands in existing floodways are more complex. Extensive hydraulic modeling of the floodway channels is submitted to the federal and state governments for approval. Approval by the funding agencies is critical before the final site selection is completed.

Project Team

To guide the planning and implementation of these projects, project teams consisting of citizens and staff from various county and municipal agencies are formed. Members are also recruited from universities, nonprofit organizations, and consultants. An informal inter-agency project support group (Figure 6) leads the overall project. This organizational arrangement promotes greater integration of agency functions, stakeholder participation, and community based initiatives.

Environmental, engineering, urban planning, and project management staff are all needed for the project. Support group members provide different functions. For example, the Department of Environmental Resources provides technical support in the areas of engineering, project management, and funding coordination; the Maryand National Capital Park and Planning Commission provides community planning and liaison; and town officials, a community development corporation, and various citizen groups represent public and private interests, respectively.

Costs

To date, three small pilot bioretention streetscape projects have been completed in the Port Towns. Drainage areas for all of the bioretention projects were less than one acre in size and 100% impervious. The average cost of these bioretention retrofit projects (Table 3) amounts to $44,000 per acre of impervious drainage area.

Two shallow marsh extended-detention ponds have also been constructed in the Port Towns at an average cost of

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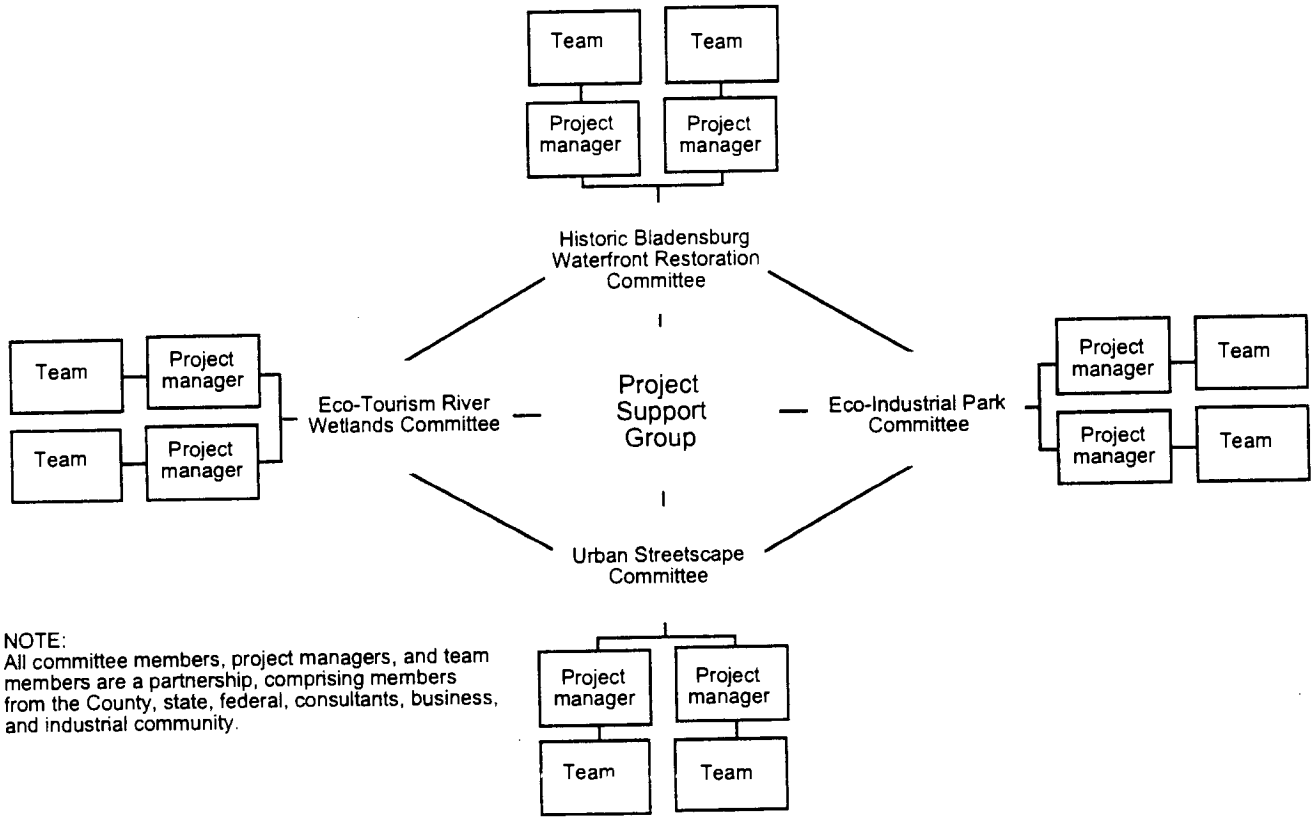

Figure 6. Port Towns revitalization organizational structure.

Table 3. Stormwater Construction Cost

Project Name	Drainage Area (acre)	% Imp.	Total Cost	Cost/acre Imp. Area
Bioretention Pilot Projects - Commercial Land use				
Chesley	.94	100%	$27K	$29K
Colmar Manor	.65	100%	$44K	$67K
Bladensburg	.66	100%	$23K	$35K
Shallow Marsh Pond - Residential Land use				
Bladensburg	230.00	55%	$395K	$3.1K
Cottage City	38.50	69%	$91K	$3.4K

$3,250 per acre of impervious area. Costs for bioretention retrofit projects in existing urban areas are relatively high due to complexities related to limited space, intense traffic controls, and presence of existing utilities. Although average costs for similar BMPs in the County over the last 6 to 8 years exceed those of projects in the Port Towns, such costs should be comparable in the longer term.

The cost of river wetlands reconstruction is expected to be much higher than the norm for the County, due to existing flood levees. An assessment of all the Port Town retrofit opportunities and their associated costs is being prepared.

Conclusion

The Port Towns is one of the first communities to envision that stormwater retrofit projects can play a leading role in urban revitalization. Stormwater retrofit projects comprise a new approach to publicly funded infrastructure rehabilitation projects that help renew old urban areas. Due to the comprehensive nature and strong community involvement of the Port Towns revitalization initiative, the

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projects have drawn considerable funding support from federal and state agencies.

Pilot stormwater projects were built within a 12-month period to demonstrate tangible results and achieve political and public support for revitalization projects. These projects demonstrate that stormwater projects constitute an effective tool to retrofit and improve the appearance and image of existing urban communities. Planning and design of more ambitious and complex projects are already underway, including reconstructing wetlands in the Anacostia floodway, streetscaping major transportation corridors, and cleaning up the industrial park.

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Tollgate Drain - An Innovative Approach to Stormwater Management

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The Tollgate Drainage District is a dedicated drainage district that was established, in the late 1800s to be utilized by Lansing Township, the City of Lansing, and Ingham County. The Tollgate Drain has used the City of Lansing's combined sewer system as its outlet since the early days of the City's sewer system. As part of a recent 30-year plan developed by the City of Lansing to control combined sewer overflows to the Red Cedar and Grand Rivers, the Tollgate Drainage District, under the direction of the Ingham County Drain Commissioner's Office, was mandated to implement a combined sewer separation project. Now the City of Lansing sends sanitary water to their wastewater treatment plant, while the stormwater is diverted to a wetland park detention basin and a series of detention ponds incorporated into the reconstruction of the local Groesbeck Municipal Golf Course.

Objectives

The Ingham County Drain Commission recognized the importance of redeveloping property for the dual purposes of the neighboring Fairview Park and for stormwater management. The primary goals of the Tollgate Drain project were to eliminate combined sewer overflows through sewer separation and develop a wetland ecosystem that improved storm water quality, while also meeting the aesthetic needs of Fairview Park and the Groesbeck Municipal Golf Course.

The Project

In accomplishing these objectives, the Tollgate Drain project created a wetland to act as a natural filtration system for stormwater runoff. The wetland helps maintain water quality by removing nutrients and sediments In the water. It involves the development of a stormwater separation, retention, and recharge system. The end result is a state-of-the-art urban wetland management system that uses innovative and cost-effective methods of water management.

The 210-acre Groesbeck neighborhood had a one-pipe sewer system which was built in the 1950s. This system was recently incorporated into a new two-pipe combined sewer system. One pipeline transports household waste to the C4 of Lansing's wastewater treatment plant while the other pipe, containing stormwater, is diverted. A traditional method of stormwater disposal is to drain it into the nearest river. Using an innovative method, Tollgate Drain diverts stormwater to the lowland area of Fairview Park, where it is naturally cleansed of non-point source pollutants and then recharged into the air and the ground. The water is also used to irrigate the Groesbeck Municipal Golf Course.

Most stormwater systems incorporate little if any nonpoint source pollution abatement. Stormwater picks up road oil, organic debris, fertilizers, salt and other forms of pollution as it makes its way through the stormwater system to the rivers and their tributaries. For this reason, the Tollgate Drain project is unique. Unlike other stormwater systems, it does not outlet to a river and it has nonpoint source pollution abatement properties.

Seven-Steps of the Tollgate Drain

Overall, the project consisted of seven key elements:

Step 1: Develop a catch basin maintenance plan.

Step 2: Create a filter chamber to act as a secondary cleaning chamber.

Step 3: The wetland design.

Once the stormwater reaches the wetland, it runs through mechanical oil skimmers and sediment traps which remove petroleum products and excess sand and mud. Peat-sand and limestone are used as filters in the system. Their high phosphorous (P), biological oxygen demand (SOD), pH (acidity), and pathogen removal capabilities, coupled with simple design, low maintenance, and affordability make them an attractive method. The wetland provides a variety of functions such as flood control/water storage and filtration of pollutants, and it creates eleven acres of wildlife habitat. From the Fairview Park wetland the water travels through a pipe under Wood Street to Groesbeck Municipal Golf Course where it flows into additional wetland detention ponds. This evaporates some

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of the water and allows the suspended sediments to further settle out. At this point, the golf course has the option to use this water for irrigation.

Step 4: The holding ponds on the golf course.

Step 5: The ultimate discharge to the City sewer at a restricted rate.

Step 6: A proactive public outreach program within the drainage district to inform and educate the district on their role in this project

Step 7: A public outreach program with a broader perspective for the community at large.

Educational Aspects

The Drain Commissioner's Office ran an extensive public outreach and education program for this project. All residents received a survey and a door-to-door visit to discuss sump pump connections and elimination of illegal storm water cross-connections to the sanitary sewer system.

An on-site office staffed with Ingham County Drain Commissioner representatives was available throughout the construction phase of the project with a hot line so that residents' concerns and questions were dealt with immediately. A door-to-door follow-up survey was conducted to obtain feedback and continue the urban storm water education.

The overall maintenance of the project depends on how the residents of the district take care of it. Residents are encouraged to participate in the success of the project by tailoring their daily activities to decrease the amount of pollution, and therefore decrease the maintenance costs. Dumping oil, pet waste, cigarette butts, or other garbage, and blowing grass clippings and other yard waste into the streets increases the number of times the catch basins will have to be cleaned out. The use of fertilizers, pesticides, and herbicides on lawns brings pollutants that can also increase the number of times filters in the system will have to be cleaned out and replaced.

Project Challenges

One of the main obstacles to overcome in the project was the "ownership" of Fairview Park. Technically, the park was owned by the State of Michigan, located in Lansing Township, and maintained by the City of Lansing as a Lansing Park. A lengthy battle over the use of the land caused uncertainty among the residents and between the different governmental agencies. Before the drainage district could proceed with design plans for the project these parties had to come to an agreement. This was the most difficult phase of the entire project and today, all parties are in agreement and cooperating fully with the Drain Commissioner's office to make the project a success.

A design challenge was to determine a cost-effective outlet for the storm sewer discharges so the sewer separation could be completed. The district is surrounded by a developed City on three sides and Groesbeck Municipal Golf Course on the west. A conventional piped storm sewer outlet to the Grand River would have had to extend over one mile through a densely-developed residential area. This option cost was in excess of $15 million. Three other routes were examined ranging in costs from $15 million to $20 million. This project, chosen instead of those options, cost $6.2 million.

Innovations/Benefits

- The key to the savings is putting nature to work. The storm water is pumped to what was once a little-used 11-acre nearby park.
- An oil and grit chamber was used to trap any oil washed into the storm collection system.
- Contaminants settle out of the water into ponds. As the water moves through connecting channels, limestone rocks buffer the acid it contains. Fast-flowing streams increase oxygen and encourage the growth of pollution-eating microbes. A peat bog filters out fertilizers and pesticides.
- The water enters a wetland where it evaporates or is recycled for more treatment. Additional water goes into water hazards at the Groesbeck Municipal Golf Course and is used for the golf course irrigation system.
- Numerous trees, plants, and grasses were planted in the system to trap contaminants. The species were selected to maximize evapotranspiration.
- The final selected system design was $6.2 million, about one-half the cost of the other options. Tollgate Drain not only saved millions of dollars, but it is a prototype for environmentally sound water management practices.
- The system not only keeps basements from flooding, but cleans the water so it can be used for irrigation at the nearby Groesbeck Municipal Golf Course.
- Storm water is managed on-site, rather than being exported.

Conclusions

In the future, direct river discharges will more than likely have to be rebuilt to accommodate non-point source pollution abatement, similar to this project, before discharging into the river. In this sense, the Tollgate Drain is ahead of its time. The Tollgate Drain involves the creation of a new wetland ecosystem designed to naturally clean and recharge the neighborhood's storm water.

Aside from the physical challenges, the success of this project is dependent upon the cooperation of the City of Lansing, Lansing Township, and the State of Michigan. But, most importantly, the future success of this project lies in the hands of the residents within the Tollgate Drainage District.

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A Stormwater Banking Alternative for Highway Projects

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Purpose

The purpose of this paper is to make others aware of the approach the Delaware Department of Transportation (DelDOT) is taking toward stormwater quality management. This is a discussion paper that looks at the costs and savings of the stormwater banking approach adopted by DelDOT and provides useful information regarding program implementation for anyone considering initiating a similar approach.

Introduction

In 1996, in response to impacts from water quality control laws at both the state and federal levels, a memorandum of agreement (MOA) was drawn between DelDOT and the state's stormwater regulatory agency, the Delaware Department of Natural Resources and Environmental Control (DNREC)[1]. This MOA establishes criteria whereby DelDOT can consider a regional alternative to the on-site approaches set forth by statute. It allows DelDOT to mitigate the water quality impacts associated with highway projects elsewhere in a watershed if on-site options are not practicable. The agreement is for water quality control only. Increases in peak flow rates associated with highway development must still be controlled on-site. While the approach deviates from the on-site approach stipulated in state regulations, both parties to the agreement believe it is consistent with state and federal water quality goals.

The MOA is often referred to as the stormwater "banking" agreement. The "banking" term is used because implementation of water quality control best management practices (BMP) is tracked using a system of credits and debits. Credits and debits are accrued by watershed. Watershed delineations can be nebulous, but in Delaware DNREC officially delineated and defined 41 watersheds[2]. The MOA is consistent with this delineation. Each watershed represents a separate bank account. Water quality credits and debits from multiple highway projects may be applied to each watershed. That is, some projects may be built without providing water quality control by accruing a debit to the watershed while other projects that do build BMPs accrue credits.

This balancing of water quality credits and debits allow a more flexible implementation of water quality BMPs for each watershed. Rather than siting BMPs within the limits of specific projects, DelDOT may look anywhere in a regional watershed. It is believed this approach will direct the limited funding available for BMP implementation toward locations most conducive to water quality treatment. In this way, Delaware can realize a more cost-effective and environmentally beneficial infrastructure of water quality treatment measures.

Presently, DelDOT is the only agency within Delaware to use such a system of stormwater banking. So balancing or trading of water quality credits is only between DelDOT projects. It is hoped that other agencies will develop similar agreements so that trading of water quality credits may be conducted between multiple users. This could include both private and public entities.

Local Factors That Enable MOA Development

For those contemplating development of a similar agreement, it is useful to understand the regulatory climate in Delaware that allowed development of the MOA.

Program Delegation

EPA delegated the National Pollutant Discharge Elimination System (NPDES) permit program to the state DNREC. It also delegated the Coastal Zone Management Program (CZM) to the state. Being a delegated agency of EPA for both the NPDES and CZM programs gives the State of Delaware some latitude in implementing its surface water quality control programs.

Also, DelDOT's situation is unique among state DOT's in that DelDOT is delegated by DNREC to administer its own stormwater management program. Embodied in the state stormwater management law is a provision that allows delegation of program functions to other state agencies that can demonstrate the technical and financial abilities to implement this program. DelDOT sought and received program delegation in 1991. This gave DelDOT the ability to design, review, and permit its own projects for stormwater management.

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Having control of its own stormwater permitting program affords DelDOT some freedom in program implementation. In fact, DNREC encourages each delegated agency to implement policies and procedures that address local needs and initiatives. This MOA stems from that philosophy and was developed to offer an alternate means of achieving the state water quality control goals in a way that considers the limitations of highway projects.

Need For MOA

To understand the need for this MOA one needs to understand the traditional site-specific approach toward stormwater management stipulated in the state regulations and its impact on DelDOT.

Under the state law, DelDOT is required to implement stormwater management controls on every project involving land disturbances of 5,000 square feet or more. Stormwater ponds and other control measures are required for drainage areas measuring only fractions of an acre. Highway projects, being long and linear, cut across multiple watersheds, sub-watersheds, catchments, and subcatchments - requiring multiple stormwater ponds on every project. DelDOT has found this site-specific requirement leads to a proliferation of small expensive stormwater management ponds.

One project in particular illustrates this fact. State Route 1, North of Smyrna, Delaware, consists of a 6 mile stretch of 4-lane dual divided highway on a new alignment[3]. The preliminary project plan submittal proposed 43 ponds to manage the runoff from every drainage area affected by the project. Each drainage area was on the order of 1-2 acres. Later this number was reduced to 13 ponds by combining the runoff from multiple drainage areas. But this is still a large number, especially since it only addresses the needs for one project. When the whole transportation system was considered, it became evident to DelDOT that the site-specific approach to stormwater quality control would lead to an unsupportable expansion of public infrastructure.

Somewhat worse situations arose on widening projects where multiple stormwater ponds had to be fit into previously developed landscapes. On projects such as the widening of Naamans Road, DelDOT actually purchased homes to make room for stormwater ponds[4]. The cost included the fair market value of the homes, relocation of the residents, demolition of the existing structures, and construction of the ponds. In one case, the cost of a pond to treat less than a 2-acre drainage area exceeded $300,000.

Because of the SR-1 and Naamans Road experiences, in the fall of 1994 DelDOT took an inventory of all stormwater management facilities planned for construction over the next 6 years (FY95-2000). An examination of project plans indicated that at least 114 new stormwater management ponds were in some stage of design or construction. Of those facilities, a representative sample of 37 ponds examined to determine some average conditions that could be applied for cost estimating purposes:

- The average land area required per pond — 0.75 acres
- The average volume required per pond — 3,000 cubic yards
- The average depth of the ponds — 4 feet
- The average cost of real estate — $25,000/acre

From these parameters, stormwater pond construction was estimated by applying the mean cost of DelDOT construction pay items. The cost per pond came to $85,225. This included real estate acquisition, design, and construction costs. The projected construction cost for all 114 ponds was estimated at $9,715,650.

The inventory and cost estimate also included other types of stormwater management practices planned for implementation on DelDOT projects for this time period. Included were infiltration trenches, biofiltration swales, and sand filters. Their costs were estimated to be $497,460. The total construction cost of all stormwater management practices for the 6-year period (FY1995-FY2000) was estimated at $10,213,109.

Compared to the $463,349,000 Capital Transportation Improvement Program Budget for this period, the cost of stormwater quality control implementation amounted to only 2.2% of the budget. But while 2.2% may seem small, it still amounts to a substantial investment in public infrastructure. And certain recent projects raised doubts as to whether the limited funds available were being used in the most effective manner.

For example, the Lancaster Pike widening project included 816 linear feet of sand filters to treat the runoff from about two acres of roadway pavement[5]. Of the two acres, only about one acre was new pavement. The cost for the filters was $326,400. It seemed exorbitant to managers at DelDOT, but at the time of design no alternative existed. The project's steeply sloped and high-cost real estate surroundings were not conducive to less-expensive options and the on-site requirements in the regulations obligated DelDOT to provide water quality control for this drainage area. This project caused DelDOT managers and members of the public to question whether the high cost of water quality control was worth the seemingly miniscule environmental benefit which was difficult to measure. Because of this and several similar situations on other projects, DelDOT and DNREC collaborated to devise a better way.

The Stormwater Banking Concept

The quality of water in a stream depends on many factors - just one of which is stormwater runoff. When considering non-point sources of pollution, it is commonly thought that all land surfaces contribute some degree of pollution. And the relative amounts of pollution will vary naturally

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within watersheds from one sub-area tc another. In this context, the quality of stormwater runoff from any single sub-area is not a significant determinant of stream water quality. Pollution levels then can be increased or decreased from one area to another with no adverse affect to stream water quality. As long as the cumulative impact to the stream stays the same, or is reduced, stream water quality will be preserved or even enhanced. The MOA is based on this concept of balancing the levels of pollution from multiple sub-watersheds.

Specifically, the MOA allows DelDOT to provide stormwater quality.controls at an a tentative location in the event the implementation of similar controls at a specific project site is not practicable. The measures installed at alternate locations must provide stormwater quality treatment for an equivalent amount of highway runoff as that going untreated at the project site. In choosing alternate locations, preference is given to sites within the same watershed as the project. Projects requiring water quantity control must still address it on-site. Ways of providing water quantity control without building a pond are discussed in appendix 'A' of the MOA.

This concept of balancing stormwater treatment from one area to another, literally treating some areas while letting other areas go untreated, is often referred to as stormwater "banking". And, as is the case with Delaware's MOA, the concept normally uses an accounting system of credits and debits to track the overall level of water quality control implementation in each watershed - hence the term "banking".

Stormwater "banking" offers an alternative to the site-specific approach by helping to facilitate a regional planned approach to stormwater quality management. Regional planning for water quality control involves prioritizing the various water quality treatment needs in each watershed and targeting implementation of control measures in the locations they will do the most good. In theory, this should minimize the overall number of stormwater management measures and maximize their cumulative effectiveness. In this way, taxpayers should receive the greatest return on their investment in public infrastructure designed to treat highway runoff - both in terms of initial construction costs and long term maintenance.

This stormwater "banking" concept is not particularly new or original. Similar approaches are frequently employed on projects all over the country. But it is seldom well documented and is often viewed as bending the rules. This MOA formalizes the criteria by which DelDOT will determine compliance when it is not practicable to manage stormwater quality "on-site". And it validates the approach as an acceptable alternative.

MOA Triggered by Variance

It should be noted that DNREC was reluctant to depart completely from the requirement to manage stormwater quality on-site. In Section 2.2 of the MOA, it was DNREC's desire to limit the use of stormwater "banking" by allowing its use only after first exhausting all on-site alternatives. In its final form, the MOA is reserved for projects located in areas that pose difficult site constraints or which otherwise offer little opportunity to implement permanent water quality control measures on-site. The terms of the MOA may be invoked only through the granting of variance in accordance with Section 3.3 of the Delaware Sediment and Stormwater Regulations. Variances may be granted only after demonstrating that exceptional circumstances exist at a project site which would result in unnecessary hardship and not fulfill the intent of the regulations.

Site Selection

Selection of appropriate sites for stormwater banking is accomplished through guidance provided by DNREC watershed managers. The MOA encourages a collaborative effort in selecting a site.

Section 1.2 of the MOA defines 41 regional watersheds which is consistent with the delineation established by state and federal water resource managers (see the-draft Delaware Wetland Banking Agreement)[2]. It is preferable under the MOA to mitigate the water quality impacts from a project within the same watershed. However, if the committee of resource managers established under Section 3.4 determines it is appropriate to mitigate outside the watershed, then this option may also be considered.

Types of Water Quality Control Alternatives Allowed

Section 3.3 of the MOA lists the alternative types of water quality treatment methods available to DelDOT. The goal of this section is to encourage the use of alternative water quality control methods that best meet the water quality control needs for each watershed. The available options focus on protecting key natural areas such as streams and wetlands. Other banking agreements reviewed emphasized providing only stormwater management ponds or infiltration measures[6]. DelDOT and DNREC felt it was important to encourage wetland creation, restoration, and enhancement as a water quality improvement measure. The list includes: source controls, removal of existing pavement, reforestation of cut woodlands, replacement of riparian vegetation, retrofitting existing stormwater ponds, removal of illicit connections. The list itself is not meant to be all-inclusive. All reasonable water quality improvement techniques will be considered under this MOA provided they help meet the water quality goals for the watershed being considered.

One alternative that was considered, but later ruled out was conservation/preservation easements. This option would have allowed DelDOT to purchase the development rights to lands deemed worthy of protection such as upland forests which are extremely important from a water quality perspective. Unfortunately, DNREC felt this option did not mitigate increased pollution. As they saw it, preservation easements only maintain the status quo. DNREC

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argued that if DelDOT were allowed to increase pollution at one location in a watershed, then an equivalent amount needed to be reduced elsewhere. Preservation easements did not meet that test.

Accounting System

Section 3.5 establishes the accounting procedures for water quality debits and credits. They are accrued by watershed and each watershed can be thought of as a separate account. Currently, DelDOT is tracking the number of credits and debits using Microsoft Excel software.

Counting credits and debits seems like it ought to be a fairly easy thing, but it becomes very complicated if certain factors are considered such as whether the land treated is impervious, farmland, subdivision, or forest. Questions arise such as, is it fair to give equal credit to a measure that treats runoff from fallow fields as one treating roadway runoff. Other complicating factors include whether the treatment measure is in the same watershed as the project. Should it be given equal credit? Maryland's agreement attempts to consider these factors. DelDOT decided this approach was just too cumbersome for our purposes. We limited the credits to the actual acreage of impervious surface treated. Even with this simplification, a supplemental worksheet was prepared at the request of project designers struggling with the accounting of water quality credits and debits.

Modification and Termination of MOA

It was important that both parties have the ability to alter the agreement. Since the regional concept had not been tried in Delaware previously, neither party was quite sure how well the concept could be implemented with this simple agreement. It is expected that the MOA will need updating from time to time as our understanding of the regional stormwater management approach matures. Therefore, Sections 3.4 and 3.7 of the MOA allow for modifications upon written agreement of both parties.

Both parties acknowledge the agreement relies heavily on a mutual understanding of each agency's needs and limitations. It will succeed as long as conditions exist which foster a cooperative spirit. This could change over time because of political or personnel changes which might result in philosophical differences. Should the relationship degrade, the MOA may be terminated upon written notification by either party in accordance with Section 3.8.

Funding of Stormwater Banking Projects

The MOA itself does not stipulate the way DelDOT will fund stormwater banking projects. Funding opportunities will vary over time so there seems no reason to create binding arrangements in the agreement. The agreement does, however, establish a funding time frame under Section 3.5. This Section obligates DelDOT to fund a banking project to mitigate for previous debits within three years of first using the MOA.

There is no expectation that federal or state grants will help fund this program. Rather, funding will be part of DelDOT's operating and capital improvement budgets. However, DelDOT is investigating other funding options. Current funding options being used or considered include the following alternatives:

1. Banking projects funded as component of highway contract
2. Percentage of contract cost held in escrow from multiple projects or programs
3. Public-private partnerships
4. Public-public partnerships

Under item (1), if banking can be accomplished within the limits of an existing project, the costs can be made part of that project. Normally, this is done when an opportunity exists to manage the runoff from more land than what is required under the project, such as adjacent existing highway. In this way, water quality credits are accrued for the watershed in question. Future projects then may be built without stormwater quality controls by taking debits against the credits accrued by earlier jobs.

Under item (2), DelDOT will hold a certain percentage of program funds aside for stormwater banking implementation. For instance, this is being done with DelDOT's Pavement Management Program which funds pavement overlays, shoulder paving, and minor (1'-3') lane widening projects. In FY99, 1% of program funding ($320,000.) is being set aside to address water quality concerns arising from this program. The amount set aside is based on an estimate of additional acreage of impervious surfaces created under the program. Mitigation efforts will be focused in one or two high-priority watersheds to balance the impacts of many projects from multiple watersheds around the state. Under this scenario, highway projects may be started prior to actually having a banking project initiated. So long as the funding is available, the impacts from earlier projects can be mitigated within the 3-year time limit.

Item (3) has been discussed but no agreements have been reached as of this writing. However, it is envisioned that a private developer or group of developers could partner with DelDOT to build one or more regional facilities that manage the runoff from both private and public land. There are multiple ways to fashion a partnership under this scenario. The main bargaining chips include land, design services, construction, and future maintenance.

The public-public partnership under item (4) presents itself in locations where multiple public agencies share real estate, but maintain separate operating budgets. DelDOT has identified several locations where other state agencies, local governments, and school districts may partner with DelDOT to share the costs of building and maintaining a stormwater banking facility. No agreement has been signed to date, but several are in draft stages.

Both options (3) and (4) rely on equitable distribution of costs. DelDOT is settling on a formula of distributing costs

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based on percentage of land contributing runoff. The cost to each partner is the total cost multiplied by each partners respective acreage of land as a percentage of total acreage contributing runoff to the facility. This formula is used to divide construction costs. It can also be used to determine each partner's annual share of maintenance costs.

Property Acquisition Concerns

It remains to be seen what authority DelDOT will be able to exercise in acquiring property for stormwater projects when it has to mitigate for highway jobs located a considerable distance away. There was concern that DelDOT would never be able to settle on a site because property owners would argue that we could always locate it somewhere else. In his legal review, the Deputy Attorney General (DAG) felt that DelDOT would enjoy all the same authority we have now. That is, if we needed property for stormwater management purposes we could obtain it either voluntarily or through invoking the state's right of imminent domain. The DAG's opinion was that as long as we can show that the sites we pick are the most practical and feasible locations we would be justified in the taking. He did not think we would need to prove the chosen sites are the only feasible locations[7].

Cost Comparison on Porter Road Project

Phase I of the Porter Road widening project serves as a good example to illustrate the potential savings of the regional approach allowed under the MOA. The project begins at the intersection of Route 896 and extends 2.225 miles East to the intersection of route 72. It involves the widening of the existing 18-ft roadway to a variable width of 48 - 60 ft. The project has outfalls to three of the 41 watersheds defined in the MOA. Within those watersheds, 13 sub-areas were identified as requiring separate stormwater management measures. The initial design was submitted with 13 stormwater management ponds to control the increased peak rates of runoff and non-point source pollution associated with the roadway project for each of those 13 subareas. The proposed measures would treat the runoff from only those areas within the immediate project limits. Their design was typical of small-scale stormwater ponds, lacking in aesthetic appeal and marginal in the overall water quality benefit to downstream areas. The estimated cost of this site-specific approach was in excess of $1 million.

Later it was determined that the MOA criteria would offer a better alternative for this project. The design proposed building wetlands instead of stormwater management ponds. The revised design included the following components:

- Wetland creation in the headwaters to Belltown Run to prevent downstream flooding and improve water quality,
- Restoration of 1 acre of previously filled wetland at the Porter Road Belltown Run crossing restoring flood plain storage and stream habitat,
- Retrofitting of an existing county owned stormwater pond to incorporate water quality control components (i.e. with extended detention), and
- Enhancement of an existing degraded wetland at the Porter Road and Route 72 intersection which involves the eradication of phragmites and creates wetlands in the upland areas adjacent to a narrow band of existing wetlands.

In addition to treating the runoff from the Porter Road project, an additional 25 acres of existing roadway was afforded water quality treatment that accrued as credits in the Christina River watershed. These credits are being used to balance the water quality impacts from the Salem Church Road project in the same watershed. The cost of this approach is estimated at $1.2 million. However, substantial savings will be realized when future projects make use of the credits afforded by the Porter Road Project.

This comparison illustrates the potential economic savings that can be generated through use of the MOA but DelDOT believes the measures installed under the MOA are also more effective from an environmental standpoint. The larger scale of these facilities allows more innovation in design resulting in many secondary benefits in terms of wildlife habitat, aesthetics, and public acceptance.

Consistency with Federal Surface Water Quality Control Programs

It was DelDOT and DNREC's intent to ensure the MOA was consistent with other water quality control programs at the state and federal levels. Section 2.1 of the MOA makes very general statements regarding this consistency merely to confirm that in fact these programs were considered. A more in-depth discussion of how the MOA meets the water quality requirements is provided below.

TMDL Program

Section 303(d) of the Clean Water Act requires states to develop a list of water bodies that need additional pollution reduction beyond that provided by the application of existing conventional controls. The law requires states to identify all waters needing water quality improvement. Those portions of streams not meeting designated use standards are termed "Water Quality Limited".

Water quality limited waters require the application of Total Maximum Daily Loads (TMDLs) to determine the allowable stress for each stream. A TMDL is the level of pollution or pollutant load below which a water body will meet water quality standards and thereby allow designated use goals, such as drinking, water supply, swimming, fishing, or shellfish harvesting to be achieved.

The TMDL approach to watershed management recognizes that streams have a certain capacity to carry pollution without any discernible impact to the designated use of a stream. It recognizes that restoration of stream water quality may require a balancing of pollutant loading from

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multiple sources in a watershed. The MOA is consistent with this concept and may to some extent help facilitate a system of trading water quality credits between multiple users within a watershed.

NPDES Stormwater Permit Program

Section 402(p) of the Clean Water Act establishes permit requirements for certain municipal and industrial stormwater discharges. New Castle County, Delaware was identified under the Phase I NPDES Stormwater Permit program as requiring a permit for the discharge of stormwater from the municipal separate storm sewer system (MS^4). Also, all construction activity disturbing more than 5 acres of land was identified as an industrial activity requiring a stormwater discharge permit. This legislation affected all storm drains owned and operated by DelDOT in New Castle County and also affected DelDOT construction activity statewide.

The statute mandated that owner/operators of storm sewer systems implement regional stormwater management plans utilizing a watershed approach. Stormwater banking ran be a component of such a plan.

The concept of banking stormwater quality improvement credits is consistent with the federal statutory requirement of implementing controls to reduce the discharge of pollutants from municipal separate storm sewer systems to the maximum extent practicable. The operative phrase in the statute is, "reduce... to the maximum extent practicable". Neither the law nor the regulations requires the discharge of pollutants associated with stormwater runoff to be eliminated or reduced at all cost. While the implementation of stormwater quality controls on each and every transportation project may be a desirable goal, it is recognized such a goal may not be realistic, cost effective, or practicable.

DelDOT believes it will be able to demonstrate compliance with the legal intent of the statute because the banking approach is based on the water quality control needs of the overall watershed. This is especially true if DelDOT can show consistency with the TMDL for each stream section. However, It may be possible for a citizen to lodge a complaint under the statute if there is a measurable increase in pollution at a specific site where water quality controls were determined to be impracticable to implement These types of complaints would likely come when a water quality impact is readily noticeable by the general public, such as where a storm drain discharges trash, debris, sediment and the like from a roadway onto adjacent property. This would most likely be the case on new alignments if control measures were not implemented. Improvements to existing alignments would not be as likely to generate these types of complaints because roadway type pollution would already be present. Adding another lane under a widening project is not likely to change the character of the pollutants to a great degree, although the total mass of the various pollutants may increase slightly.

For these reasons the agreement emphasizes the implementation of water quality controls onsite for major roadway improvement projects, such as new alignments. It encourages utilization of the banking concept only on more minor types of projects, such as intersection improvements and lane widening projects where it is typically more difficult to incorporate stormwater management measures. Also, a maximum debit limit of 5 acres is allowed to accumulate statewide before it must be mitigated by implementation of a water quality control project. The 5-acre limit was chosen to coincide with the 5-acre limit established in the NPDES stormwater regulations for construction activity. The law requires implementation of water quality controls when 5 or more acres of ground is being disturbed by construction. Under the agreement, DelDOT may do several small projects each disturbing a fraction of the 5 acres. But once the aggregate of all watersheds exceeds the 5-acre limit, DelDOT must undertake a project to mitigate for those cumulative impacts.

Coastal Non-Point Pollution Control Program

Section 6217 of the Federal Coastal Zone Act Reauthorization Amendments (CZARA) of 1990 mandated that each state in the coastal zone initiate a coastal non-point pollution control program. The intent of the law was to encourage EPA, NOAA, and the states to place special and expeditious attention on protecting the nations coastal water from urban sources of nonpoint pollution.

EPA excluded from coverage under Section 6217 all stormwater discharges covered by Phase I of the NPIDES stormwater permit program. That is, any stormwater runoff that ultimately is regulated under an NPIDES permit will not be subject to the requirements of Section 6217 of the CZARA once the permit is issued. For instance, discharges of stormwater from construction activities disturbing more than 5 acres of land and New Castle County's municipal separate storm sewer system were excluded from the Coastal Non-point Pollution Control Programs.

That still left several sources of pollution that needed to be addressed under the Coastal Zone Act. Specific areas affecting DelDOT included requirements to control runoff from existing roadways and bridges and runoff from construction sites that result in the disturbance of less than 5 acres of land.

The notion of building stormwater treatment measures as the only item of work was not commensurate with the mission of DelDOT which is to build transportation systems, not water quality treatment systems.

Fortunately, the Section 6217(g) Guidance encouraged a whole watershed planning approach in implementing stormwater management measures. Again, the MOA on stormwater banking is consistent with this concept and may to some extent help facilitate the process. From DelDOT's perspective, the MOA helps DelDOT justify the expenditure of transportation funds on water quality control measures for existing roadways and bridges. As discussed above, the implementation of stormwater management

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control measures in accordance with the terms of the MOA can be less expensive than a site-specific approach. There is an economic incentive then for DelDOT to undertake projects solely for the purposes of treating stormwater runoff because of the savings accrued to future roadway projects.

Section 404 Wetfands Permitting Program

The MOA allows the implementation of many alternative types of surface water quality control measures, one of which is creating wetlands. Wetland creation in areas designated as uplands will provide stormwater quality treatment in accordance with the stormwater regulations. With the US Army Corps' concurrence, it may also qualify for wetland mitigation credits required for highway projects.

In searching for a site to build a regional stormwater management facility, if is often the case that a stream or wetland is identified as the only feasible location. However, it is not usually possible to acquire permits to locate stormwater management measures in existing wetlands, nor does the MOA encourage this activity. Under certain circumstances, however, the regulatory agency may believe work in a wetland is beneficial to the resource such as by restoring a previously filled wetland. For instance, DelDOT has restored previously filled wetlands for mitigation credits on several projects. If these restored wetlands rely on surface runoff from roadways to provide the hydrology needed to support the wetland, then stormwater quality credits may also accrue under the MOA for the watershed in question.

Conclusion

The on-site approach to implementing water quality treatment measures encourages a proliferation of small, expensive, and maintenance intensive practices on DelDOT highway projects that may not offer the best solutions needed for the watersheds in question. Stormwater banking offers one possible alternative because it allows implementation of treatment measures anywhere in a watershed so they may be targeted toward the areas they are needed most.

Delaware's MOA allows a broad array of treatment methods, such as wetland creation, reforestation, and elimination of existing pavement. This is intended to encourage innovation in meeting the water quality requirements and protect key natural areas. The MOA also offers a uniform procedure for tracking water quality credits and debits accrued in each watershed. DelDOT is finding the stormwater banking approach to be more flexible and cost effective than the traditional on-site approach.

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Financing Retrofit Projects: The Role of Stormwater Utilities

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To achieve national water quality objectives, we must retrofit existing stormwater infrastructure to manage pollution in runoff. Many public works and water resources professionals have suggested that stormwater utilities are an important, if not essential, funding source for retrofit projects. We examine in this paper the role of stormwater utilities in financing retrofit projects and programs. Based on a broad assessment of the need for funding and a brief overview of the evolution of stormwater utilities, we conclude that stormwater utilities are perhaps the best institutional approach to financing retrofit programs, but that they are not a panacea. The major issues in implementation of effective retrofit programs will be economic and therefore political. Stormwater managers can help constrain and focus political debate through careful analysis.

How Much Do Retrofits Cost? What is the Need For Funding?

The answers to questions about the costs of retrofits and programs to control the quality of stormwater runoff depend on many different factors. These factors include the characteristics of runoff quantity and quality, the size of the watershed where projects are being planned, the severity of water quality problems, the water quality objectives, and the types of best management practices that are being proposed. A short, safe answer that recognizes variability among places is that programs will be expensive, ranging from tens of thousands of dollars in relatively small places to achieve modest objectives, to tens or hundreds of million dollars in larger cities with moderate to severe problems.

We provide here several brief examples of the potential magnitude of costs of retrofit programs. Our examples are by no means exhaustive and they are not necessarily representative. We have chosen published estimates or used cases with which we are familiar simply to demonstrate that experts believe costs will be significant and controversial. These examples should be sufficient to convince skeptics or individuals who have not thought systematically about the economic aspects of retrofit programs, that a critical task in implementation of a program is identification of sources of revenue.

Our examples include estimates of the costs of programs at the national level, for a watershed, for a large city, and for a small town (Table 1). In a project for the American Public Works Association, James M. Montgomery (1992) estimated the capital and operation and maintenance (O&M) costs for large and medium cities to comply with EPA's stormwater rule. Capital estimates ranged from $147 million to more than $400 billion, depending on assumptions about the level of treatment for runoff. Estimates of O&M costs ranged from $1.2 billion to more than half a trillion dollars, again depending on assumptions about level of treatment. Reasonable questions can be raised about these estimates. Some experts suggest that they are too high because advanced treatment never has been contemplated for stormwater. Other critics contend that these estimates were made primarily for political purposes and to support opponents to the then-proposed federal stormwater rule who argued that costs were prohibitive. Regardless, they are suitable for our purposes. They demonstrate clearly the need for financing and they show that the costs of programs will be controversial.

More recently, EPA modeled the Phase I Storm Water needs to inform Congress of the costs of programs to control pollutants in urban runoff. Approximately 266 Phase I stormwater permits that regulate about 850 municipalities will be issued. The Phase I needs estimates were prepared to determine the stomwater management costs that might be eligible under state revolving fund (SRF) loan programs. The SRF-eligible costs include costs for developing and implementing municipal management programs, including capital costs for structural controls and BMPs. The total modeled costs are $7.4 billion. These costs do not include O&M costs, costs of land acquisition, permitting costs, costs of developer-financed BMPs; or several other categories of costs. These estimates, which were subject to peer review prior to their release, also are significant. These esti-

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Table 1. Selected Costs for Stormwater Programs

	Estimated Costs for BMPs in Regulated Municipalities (Montgomery 1992)	
	Capital	O&M
1. Source controls	$147,100,000	$1,155,000,000
2. Increased maintenance + 1	$147,000,000	$32,607,800,000
3. Construction of moderate controls + 2	$83,139,500,000	$86,223,700,000
4. Construction of detention basins + 3	$91,130,900,000	$90,097,500,000
5. Advanced treatment plants + 4	$406,734,900,000	$542,036,700,000
	Estimated Costs for Phase I Storm Water Programs (EPA 1997)	
Modeled SRF-eligible costs	$7,400,000,000	
	Costs for Pollutant Reduction in the Menomonee River Watershed, Milwaukee, Wisconsin (WDNR 1992)	
Core (source controls)	$3,400,000	
Segment (planned, new areas)	$11,700 000	
Segment (existing areas)	$94 - $184,000:00	
Total	$110 - $200,000,00	
	Estimated Rehabilitation Costs in Indianapolis, Indiana	
Capital	$283,000,000	
Household	$54	

mates also have political dimensions; they were prepared to inform Congressional debate over funding for water quality programs.

In general, better cost estimates can be made for smaller geographic areas because site specific factors can be taken into consideration and fewer general assumptions need be made. The Wisconsin Department of Natural Resources (1992) has estimated the costs to achieve pollutant reduction objectives for the Menomonee River in Milwaukee, Wisconsin. The Menomonee River watershed is 136 square miles, is 60% urban, and contains 18 municipalities and parts of four counties. To meet ambient water quality standards, programs are needed to reduce sediment by 50%, phosphorus by 50%-70%, and lead by 35%-70%. The corresponding cost estimates for "segment" controls for existing areas of development range from $94 million to $184 million.

In many of the nation's larger cities, stormwater infrastructure has fallen into disrepair, and significant investments will be required simply to meet generally accepted engineering standards for stormwater conveyance and flood control, let alone implementation of BMPs to meet water quality objectives. In Indianapolis, Indiana, for example, a mayor's blue-ribbon panel estimated the costs to rehabilitate stormwater infrastructure at $283 million. The infrastructure includes 1750 miles of storm sewers, more than 1000 outfalls, more than 50 miles of levees, and a number of regional detention ponds. The panel did not estimate costs for programs to manage pollution in runoff.

In smaller towns, individual projects that in larger cities would be considered routine can pose significant burdens. In Vincennes, Indiana, a city with a population less than 20,000 and a median household income two-thirds of the state median, the city is responsible for pumping water from a drainage ditch over levees into the Wabash River whenever water in the ditch reaches specified elevations. The pumps are more than 50 years old and are in poor repair. The city estimates annual costs for City Ditch to be approximately $50,000, but no existing sources of revenue are available to pay for rehabilitation and related O&M costs.

In sum, while the estimated costs of retrofits vary tremendously with the scale and scope of a program, invariably new sources of revenues will be required to pay for new programs. The costs of programs are debated by officials who have responsibility for implementation of them. Stormwater utilities have emerged from these debates as the option of choice to fund new programs.

How Can Retrofit Programs Be Funded? What Are Stormwater Utilities?

Most jurisdictions historically have paid for investments in stormwater infrastructure with revenues from property taxes and other general revenues. Many, if not most jurisdictions, now rely on a variety of sources to finance comprehensive stormwater programs. Table 2 is an abbreviated list of sources of revenues available to pay for different elements of stormwater programs. One key observation from this list is that the sources of revenues most important for retrofit programs are property taxes and stormwater user charges.

Stormwater user charges or fees are charges based on some indicator or proxy for the actual volume of stormwater runoff that leaves a property. The most common type of charge is based on the amount, or square footage, of im-

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Table 2. A Functional Approach to Stormwater Financing

BMP	Option
• Watershed planning	- general revenues (property, income taxes) - stormwater user charges
• Source controls - Enforce ordinances - Development regulation	 - general revenues, stormwater user charges - plan review & inspection fees
• Maintenance (e.g., street sweeping)	- general revenues - stormwater user charges
• Capital projects - new development - retrofit existing areas	 - developer exactions, fees-in-lieu - bonds, sinking funds - general revenues, stormwater user charges

pervious area on a parcel. Other bases for stormwater charges include the area and proportion of impervious cover on a parcel, the intensity of development, and the type of land use. In some instances, an estimate of the actual volume of runoff or some estimate of the concentration of pollutants in runoff may be used as the basis of charges. Examples of rate structures are shown in Table 3.

Stormwater charges usually are administered by a stormwater utility, an administrative unit or institution established within or across jurisdictions for the purpose of managing runoff and related problems. Revenues collected by utilities are placed in separate enterprise funds or accounts and can be used only for stormwater related expenditures. The first stormwater utilities were established in the mid-1970s, primarily to provide sources of revenue for maintenance of stormwater infrastructure. Since the 1970s, the number of utilities has grown tremendously, fueled in part by the efforts of stormwater managers desperate for funds to do their jobs.

Since the 1980s, as part of efforts to develop new sources of revenues for stormwater programs, a number of surveys of stormwater utilities have been completed. Table 4 is a summary of some of the results of these surveys. Important observations include:

- Average annual charges for residential property owners range from $15 to $130.
- Average annual charges have increased over time.
- Stormwater charges are the source of most revenues for most stormwater utilities.
- The proportion of charges from different types of property varies considerably.
- Total revenues from charges are significant and increasing.

What Are the Advantages and Disadvantages of Property Taxes and Stormwater User Charges? Why Has the Number of Stormwater Utilities Increased?

Stormwater utilities and user charges offer a number of advantages over property taxes, the main alternative, although taxes are preferable by some criteria (Table 5). It is useful to consider the drawbacks of charges first. Stormwater user charges are more difficult and costly to implement than are taxes because institutions and procedures to levy and collect taxes are already in place. User charges are not deductible from federal and state income taxes, and they are not elastic. Property taxes, on the other hand, are deductible, and revenues from them increase as property values appreciate without explicit decisions by officials to increase rates or levies. Revenues from user charges increase only if officials vote to increase rates.

Despite these disadvantages, reliance on stormwater user charges is increasing, partly because user charges are perceived as a more stable source of revenues. As noted above, revenues from charges are placed in enterprise funds and can be used only for stormwater related expenditures. Funding from general revenue sources like property taxes is never secure because of fierce competition among political leaders and program managers for scarce dollars. Under property tax systems, stormwater managers often cannot count on budget allocations, do not have as much control over their budgets, and cannot plan as well.

Perhaps the most important reason that the number of user charge systems is increasing is that property owners believe charges are fairer. Impervious area - the basis for most stormwater charges - can be measured and is a reasonably objective measure. The idea that property owners pay in proportion to the measured amount of hard surface on their property seems fair. Property values, conversely, are unrelated to the problem of runoff and perceived as highly subjective. Many surveys suggest that property taxes are the least popular form of tax.

A final reason that charges are preferable to taxes is that they provide incentives for property owners to reduce the amount of impervious area on their property and thereby reduce volumes of runoff. Depending on how credits against charges are structured, they also can provide incentives for on-site management

Local officials routinely consider these tradeoffs when evaluating sources of funds for new programs like retrofit programs. Because perception of fairness is such an important factor in public finance, it is useful to elaborate on the issue of equity.

Who Pays More Under Property Tax and User Charge Systems?

Although charges typically are perceived as fairer than property taxes, this does not necessarily mean that any

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Table 3. Utility Rate Structures in Austin, Cincinnati, and Ft. Collins

	Austin, Texas		Cincinnati, Ohio	Ft. Collins, Colorado		
Land Use Categories	Rate Factors	Rate Categories	Intensity of Development Factors	Basic Category Development	Runoff Coefficient	Rate Factor
• Undeveloped	.10	•Class A Residential (< 10,000 sq. ft.)	.25	•Very Light	.00-.30	.25
• Residential	.40	•Class B Residential (> 10,000 sq. ft.)	.20	•Light	.31-.50	.40
• Nonresidential	.80	•Class C		•Moderate	.51 -.70	.611
		- Commercial	.85	•Heavy	.71-.90	.80
		- Industrial	.75	•Very Heavy	.91-1.0	.95
		- Multi-family	.60			
		- Transportation	.50	Runoff Coefficient (C)		
		- Institutional	.40	C = Percent Impervious Area x 0,95		
		- Agriculture	.08	+ Percent Pervious Area x 0.20		
		- Park	.05	+ Percent Semipervious x 0.50		
		- Undeveloped	.00	where		
		Area Range		-impervious means roof, concrete, etc.		
		Numbers	Area (sq. ft.)	-pervious means lawn, open space, etc.		
		1	0-2000	-semipervious means gravel, etc.		
		2	2001-4000			
		3	4001-6000			
		4	6001-8000			

Table 4. Overview of Selected Stormwater Utility Surveys, 1988-1996 (Ungan 1997)

Date	Survey	Range of Population Served	Range of SFR* Charges	Range of Total Utility Revenues (000)	Range of Total Revenues from Charges (000)	Range of Charge Revenues as % of Total Revenues	Range of SFR Charges as % of all Charges
1988	Stormwater Management Adminstration, Maryland Department of the Environment (MDE) (Lindsey, 1988)	20,000-684,565	$1.25-$3.63	$263-$8200	$425-$8200	78%-100%	24%-62%
1990	MDE (Update of 1988 Survey) (Lindsey, 1990)	NA	$1.07-$7.45	$75-10,471	$75-$10,471	82%-100%	15%-78%
1991	The Florida Department of Environmental Regulation (1991)	NA	$1.00-$4.50	$118-$6850	$118-$6850	19% - 100%.	NA
1992	Black & Veatch Communications (1992)	11,000-329,227	$0.24-$9.06	NA	NA	62% - 100%	NA
1992	Apogee Research Inc. (1992)	4,300-535,000	$1 -$4.50	NA	$75-$18,316	8%-100%	NA
1993	Apogee Research Inc. (1994)	NA	$0.24-$9.08	NA	NA	NA	NA

(continued)

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Table 4. Continued

Date	Survey	Range of Population Served	Range of SFR* Charges	Range of Total Utility Revenues (000)	Range of Total Revenues from Charges (000)	Range of Charge Revenues as % of Total Revenues	Range of SFR Charges as % of all Charges
1995	Delaware Survey (1995)	6000-2,000,000	$0.50-$7.16	S19.7-$21,600	NA	NA	NA
1995	Florida Association of Storm Water Utilities (1995)	6000-2,000,000	$0.50-$7.43	S19.7-$21,600	NA	NA	NA
1996 March	Raftelis (Water and Wastewater Survey) (1996)	NA	$0.15-$10.46	NA	NA	NA	NA
1996 July	Indiana University, Center for Urban Policy and the Environment (Ungan,1997)	11,141-487,779	$0.24 $10.98	$53-$28,000	$1.8-$28,000	1%-100%	0.7%-92%
1988-1996	Min:	4300	$0.15	$53,000	$1880	1%	0.7%
	Max::	3,489,779	$10.98	$28,000,000	$28,000:000	100%	92%

Table 5. Advantages and Disadvantages of Taxes and Charges

Criteria	Charges	Taxes
• Cost of implementation	-	+
• Ease of implementation	-	+
• Deductible by property owner	-	+
• Elasticity of revenues	-	+
• Stability of revenues	+	-
• Fairness		
- user (polluter) pays	+	-
- ability to pay	+	-
- Incentives for on-site controls	+	-

particular property owner will be better off under a charge system than a system of property taxes. It is useful, therefore, to examine the relative burden on property owners under the two systems. Analyses of the relative burden typically show that, to generate a fixed sum of revenues, residential property owners pay less under a user charge system than under a property tax system. Non-residential property owners like owners of commercial and industrial properties typically pay less under a property tax system. For example, to generate $500,000 in Roseville, Minnesota, residential property owners would bear 51% of the burden under a property tax system but only 28% of the burden under a user charge system (Table 6). Similar results have been reported in most jurisdictions where utilities have been considered or established. The main reason is clear: non-residential properties are highly impervious, while residential properties are only moderately impervious, depending on their density. Another reason for the difference in burden is that tax-exempt property owners like churches, hospitals, and school pay charges. For residential property owners, the benefit is partially offset by the fact that charges are not deductible. Nevertheless, they typically are better off under charge systems.

What Are Obstacles To Implementing User Charge Systems?

Stormwater utilities are an attractive source of funds for retrofit programs, and the number of utilities has grown constantly over the past 20 years. Nevertheless, there are a number of obstacles that limit their use. We believe that the main obstacles are economic and therefore political. Many people are opposed to all new taxes, regardless of whether the taxes are perceived as fair. Hence, any time a utility is proposed, property owners will debate the merits of the proposal, and political debate will occur. Two recent cases from Indiana illustrate this point well.

In Vincennes, the Mayor sought new sources of funding to pay for pumps in City Ditch. The Vincennes City Council adopted an ordinance that established a mechanism for allocating charges among property owners in the City Ditch watershed based on parcel-level estimates of runoff volumes. The Council did not, however, pass a companion ordinance to establish a volume-charge. The Mayor lost the next election, and efforts to establish the charge system have foundered.

In Indianapolis, background studies for creating a utility were completed in the 1980s, but no action to establish a utility was taken. In 1997, following endorsement by the Chamber of Commerce, a member of the City-County Council proposed a new utility. The Mayor, who had been

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Table 6. Distribution of Property Taxes and User Charges in Roseville, Minnesota (Honchell, 1986)

	Utility Charges		Property Taxes	
User/Land Use Category	Total Revenues	Percent of Revenues	Total Revenues	Percent of Revenues
1 . Residential	$148,000	28.5%	$260,000	50.1%
2. Cemeteries/Golf Courses	$4,000	0.8%	$10,000	1.9%
3. Parks	$10,000	1.9%	–	–
4. Schools	$11,000	2.1%	–	–
5. Apartments/Churches	$44,000	8.5%	$46,000	8.9%
6. Commercial	$302,000	58.2%	$203,000	39.1%
Total	$519,000	100%	$519,000	100%

elected on a pledge of no new taxes, did not endorse the utility, but did not publicly oppose it. Many citizens and some taxpayer groups opposed the proposal, as did some individual members of the Chamber of Commerce. Votes to establish the utility have been delayed because the proposal lacks the necessary number of votes.

These cases are instructive because they demonstrate that proposed new utilities will be controversial even when stormwater problems are long-standing and well known and the proposals are backed by political leaders. In many communities, political leaders are unwilling to endure the high cost of advocating new charges or taxes. Advocates for retrofit programs necessary to achieve water quality objectives must convince political leaders that the benefits of retrofit programs exceed the costs.

Stormwater managers can inform debates through careful analysis. For example, the perceived equity of a proposed system can be enhanced through careful design of the rate structure, including features such as credits for on-site controls. Experience of local jurisdictions that have successfully established utilities demonstrates that there is not a single, correct approach. Innovative applications of basic concepts can help provide funds for retrofit programs.

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Credits as Economic Incentives for On-Site Stormwater Management: Issues and Examples

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Stormwater utilities provide an institutional mechanism for incentives such as credits or reduced user charges in the implementation of onsite stormwater management. Such incentives create greater flexibility by allowing each user to chose the least-cost option—paying the stormwater utility charge or implementing onsite stormwater management. This paper provides examples of stormwater utilities with credits for onsite storm water management, including credits for peak runoff controls, implementation of water quality best management practices, and proper maintenance of onsite stormwater facilities. Also discussed are credits as economic incentives to encourage prevention or reduction of stormwater runoff problems. As economic incentives, credits must be sufficient to induce changes in behavior; however, their impact on total utility revenues must be examined carefully.

Introduction

A stormwater utility is a public utility established to provide stormwater management services. Stormwater utilities, like other utilities, rely on dedicated user charges related to the level of service provided. These user charges are usually based on the amount of impervious area on a property (i.e., a proxy for the estimated amount of runoff discharged from a property). Stormwater utility charges typically are paid by property owners and managed in a separate enterprise fund, which is dedicated to financing local stormwater management services. Most stormwater utilities are administered under public works departments or local departments of utilities that also provide wastewater or water services.

Experience with stormwater utilities has shown that they are capable of generating substantial revenues for local stormwater management programs at relatively nominal charges. Typical monthly charges for residential users range from $2 to around $6 per month. Nonresidential property owners typically pay more because their property is generally larger and developed more intensively.

Stormwater utilities offer three major advantages over financing local stormwater programs from the general fund through property tax revenues. A stormwater utility:

- Provides a dedicated, and stable source of funds for all facets of stormwater management programs (pollution prevention, capital investments, and operation and maintenance);
- Raises funds through charges based on a user's contribution to local stormwater runoff problems an approach often seen as more equitable to rate payers or the public; and
- Provides an institutional mechanism to incorporate incentives (e.g., reduced charges) for implementation of onsite stormwater management.

Overview of Credits as Incentives for Onsite Stormwater Management

The impetus for establishing credits in a stormwater utility rate structure is that a utility may achieve greater flexibility in protecting water quality and aquatic habitat in urban watersheds at a lower overall cost to the community. This greater flexibility can also help a utility lower the total costs of stormwater management for the community. A utility could also reward those users that go beyond minimum requirements in the local stormwater management code, if a credit approach is structured accordingly.

Credits are usually made available only to nonresidential property owners. For utilities where charges to residential properties account for a significant proportion of total revenues, there is less potential for the efficiency gains possible through lowering the total costs of stormwater management.

From an economic perspective, the extent to which a credit will increase the efficiency of a stormwater program

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depends partly on the conditions in which it applies. For example, if individuals who develop property are not given the option either to build stormwater management facilities and receive a credit or to pay charges and avoid building facilities, then some of the incentive effect is lost. In cases where retrofitting is desired, whether or not a credit will induce property owners to build new stormwater management facilities where none exist or retrofit existing facilities to reduce stormwater charges depends on the size of the charge and the magnitude of the credit.

Examples of Credit (Fee Reduction) Approaches

A recent survey of stormwater utilities (NAFSMA, 1996) asked utilities whether they included incentives, such as reduced user charges, for commercial and industrial properties that implement onsite stormwater management. Of the 38 utilities that responded, 71% (27 utilities) had no fee reduction. Of the remainder of (11 utilities), two major types of fee reduction approaches were reported: 16% (6 utilities) had fee reduction for peak runoff controls, and 8% (3 utilities) had fee reduction for implementation of water quality best management practices or proper maintenance of onsite stormwater facilities. An earlier report on stormwater utilities (USEPA, 1992) found over 20 utilities with various types of credits as incentives for onsite stormwater management.

Some stormwater utilities offer credits for onsite stormwater detention/retention facilities in new developments. Credits can also provide incentives for onsite stormwater detention/retention through retrofitting older dry detention basins to extended detention basins or controlling peak flows through rooftop or underground storage tanks. Examples of credit approaches for selected utilities are highlighted below and summarized in Table 1.

Gainesville, Florida

The City of Gainesville's Stormwater Management Utility provides reduced monthly fees for nonresidential properties with privately maintained, onsite stormwater management retention systems. The maximum allowable credit is 100% of the utility's "base" fee, which is based on the amount of impervious area and one-half of pervious parking areas. The percentage of fee credit is determined by the volume of onsite retention provided (detention volume is not considered since that stormwater is discharged). The required volume is determined by the 25-year, 24-hour storm. Most credits range from 15 to 35%.

Orlando, Florida

In the City of Orlando, the stormwater utility provides a lower rate for commercial and multi-family residential properties with onsite stormwater management facilities. Such properties with approved onsite retention or detention get a credit on the rate charged per ERU (equivalent residential unit). The typical rate is $66.00 per ERU. The lower rate for properties with approved onsite stormwater facilities is $38.28 per ERU. Overall, this provides a 42% credit on the stormwater utility fee.

Wichita, Kansas

The City of Wichita's Stormwater Utility offers credits only for properties with 50 or more equivalent residential units. Two credits on the drainage fee are available. First, up to 40% credit on the fee is available for detention that equals or exceeds the city's new development standards (based on 100-year design storm). Second, an 80% credit on the fee is available for retention (no runoff from site). No credits are being given because the stringent standards are difficult to achieve.

Louisville & Jefferson County Metropolitan Sewer District, Kentucky

Credits are provided primarily for commercial properties with onsite detention for control of peak flows in the Louisville/Jefferson County Metropolitan Sewer District (MSD). A range of credits is available depending on how the detention basin functions. Basins must be sized for the 2-year, 10-year, and 100-year storms and also limit discharges to the pre-development rate of runoff. Credits are available for each type of storm, with an 82% maximum credit if all criteria are met. MSD is currently evaluating how to incorporate stormwater quality measures into its credit approach.

St. Paul, Minnesota

The City of St. Paul provides a rate of discharge credit for nonresidential properties on its storm sewer system charge. For nonresidential properties, this charge is based on actual parcel acreage and a standardized peak runoff rate determined for selected land use classifications. Where the peak stormwater runoff rate is limited by onsite facilities such as detention ponds owned and maintained by the property owner, up to a 25% credit is available. A 10% credit is provided for parcels that provide onsite storage for the 5-year design storm that also limit its discharge to a maximum of 1.64 cubic feet per second per acre. An additional 15% credit is provided for parcels that provide onsite storage for the 100-year design storm that also limit its discharge to a maximum of 1.64 cubic feet per second per acre. Both new developments and redevelopment are eligible for apply for credit. Existing nonresidential properties can retrofit to provide onsite storage for the 5-year design storm and get the 10% credit. Most credits were provided in the first few years after the credit approach was established. Currently, around 3-4 credits are approved annually. In St. Paul, the credit approach increased the political acceptability of the storm sewer system charge.

Charlotte, North Carolina

The City of Charlotte provides one or more credits for commercial, industrial, institutional, and multi-family residential properties and residential homeowner associations that mitigate the impacts of runoff on the stormwater system. Eligibility for one or more credits to the service rate

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Table 1. Summary of Credit Options

Utility	Eligible Users	Basis for Credit	Design Storm	Maximum Credit	Typical Credit
Gainesville, FL	Nonresidential	Volume of onsite	25-year, 24-hour storm	100% of base fee	15-35%
Orlando, FL	Commercial and multi-family residential	Onsite retention or detention	NA	42%	42%
Wichita, KS	Properties ≥ 50 ERUs	Two credits: volume of detention or retention	1) 100-year storm 2) Complete retention	1) 40% 2) 80%	Currently no applications
Louisville-Jefferson County, KY	Commercial properties	Onsite detention of peak flows	2-year, 10-year, 100-year storms; pre-development runoff	82%	Varies with degree of control
St. Paul, MN	Nonresidential properties	Onsite detention of peak flows; acreage, peak flows	5-year, 100-year storms; release limited to 1.64 cfs/acre	10% (5-year storm) 25% (100-year storm)	Varies with degree of control
Charlotte, NC	Commercial, industrial, institutional, multi-family residential; homeowner association	1) peak discharge 2) total runoff volume 3)annual pollutant loading reduction	1) 10-year, 6-hour 2) 2-year, 6-hour 3) reduction in loading	1) 50% 2) 25% 3) 25% Up to 100%	Varies with degree of control
Durham, NC	Nonresidential properties	Pollution credits for Water quality and quantity controls	State standards for facility design; estimated pollutant removal efficiency	25%	Few applications
Cincinnati, OH	Commercial properties	Onsite retention	Limit discharge to pre-development runoff	50%	Credit never used
Tulsa, OK	Privately maintained facilities	50% greater detention; maintenance costs of onsite facilities		60%	Varies
Austin, TX	Commercial properties	Onsite detention, inspection		50%	50%
Bellevue, WA	All properties	Onsite detention; intensity of development		Reduction of one rate (intensity of development) class	Varies
King County, WA	Commercial properties	Private maintenance		Reduction of one rate class	Varies
Indianapolis, IN	Nonresidential properties	Discharge to specified streams; onsite retention or detention watershed size	Tier Two:2-, 10-, 25-, 50-, 100-year events	Tier One: 25%; ≤$50 Tier Two: 35%; <$250	(proposed)

charge is proportional to the extent those stormwater management measures address the impacts of peak discharge, total runoff volume, and annual pollutant loading from the site. Portions of the service rate charge are available for credit as follows: up to 50% for reducing peak discharge from a 10-year, 6-hour storm; up to 25% for reducing total runoff volume from a 2-year, 6-hour storm; and up to 25% for annual pollutant loading reduction. Each credit allowed against the service charge is conditional on continued compliance with the Charlotte Mecklenburg Land Development Standards Manual and may be rescinded for noncompliance with those standards. If 100% credit is given, the affected property will receive no stormwater service charges.

Durham, North Carolina

The City of Durham provides up to a 25% pollution credit on the stormwater utility fee for selected structural stormwater controls on nonresidential properties. Currently, the maximum pollution credit goes to standard basin designs that are identified as achieving maximum pollutant removal efficiency in state performance standards. For other structural controls in the state's standards, the city's pollution credit will be linearly variable, with no credit given for a removal efficiency of 0% of total suspended solids to a 25% credit for a removal efficiency of 85% of total suspended solids. The city recently approved sand filters in addition to the approved onsite basin designs, but no pollution credits are established yet for sand filters. Durham receives few applications for credits.

Cincinnati, Ohio

The City of Cincinnati's Stormwater Management Utility offers a credit for commercial properties that install onsite

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retention that goes beyond normal building requirements (i.e., limit discharge to pre-development level of runoff). Such properties can apply for a credit of up to 50% on the utility's storm drainage service charge. This credit has never been used in Cincinnati.

Tulsa, Oklahoma

Under the City of Tulsa's stormwater drainage system service charge, credits are provided for private maintenance of approved onsite detention or retention facilities. An approved onsite facility must provide at least 50% more detention than required by the city. The amount of credit varies based upon the estimated maintenance costs if the city were providing the maintenance. The maximum credit is 60% of a property's annual stormwater charge. This maximum was established at 60% because around 60% of the stormwater utility budget in Tulsa goes to maintenance. Upon inspection, if an onsite facility is not performing adequately, then the property owner must pay the typical stormwater drainage service charge.

Austin, Texas

The City of Austin's Drainage Utility provides a 50% credit on the drainage fee for commercial property owners that construct and maintain approved onsite detention facilities. The city inspects these onsite facilities annually to ensure proper maintenance.

Bellevue, Washington

The City of Bellevue Storm and Surface Water Utility provides a credit on its storm and surface water drainage service charge for approved onsite detention facilities. This credit has worked well to get approved detention facilities built on large residential and commercial plats. Bellevue's utility rate structure classifies each property according to its percentage of developed property (from undeveloped land to very heavy development). A reduction of one intensity of development classification is provided for installation and maintenance of approved onsite detention facilities. This reduces the rate (based on the intensity of development classification) and the storm and surface water drainage service charge for such properties.

King County, Washington

Under the new King County Surface Water Drainage Design Manual, any development of parcels with over 5,000 square feet of impervious area must provide onsite detention/retention. For commercial properties, King County provides a credit through a reduction of one rate classification for the utility fee for private maintenance of an approved onsite detention/retention facility. The facility must be built to code and meet King County maintenance standards.

Issues in Establishing Credits for Onsite Stormwater Management

Like stormwater utility charges, there is no "correct" method for establishing credits. Each utility must consider local stormwater management goals in deciding whether to incorporate such incentives into their utility rate structure. The amount of impervious area on a property is usually the basis for stormwater utility charges. The quantity of stormwater runoff is generally the rationale behind charging property owners for stormwater management services (e.g., a user-pay approach). The adverse environmental impacts of urban runoff are related to both stormwater quality and quantity. To date, few stormwater utilities have attempted to incorporate measures of the quality of runoff as a basis for utility charges. Additionally, few utilities incorporate site characteristics other than impervious area (e.g., slope and soil characteristics) that also influence the adverse impacts of runoff. These factors may be important in setting charges and credits to induce the expected behavior of choosing the least-cost option. On the other hand, if stormwater quantity (as measured by the amount of impervious area) is closely correlated with adverse impacts of runoff related to both stormwater quantity and quality, the amount of impervious area may be a sufficient basis for setting charges that create the desired incentives.

Although credits must be sufficient to induce changes in behavior, their impact on total utility revenues must be examined carefully. An approach that gave large credits for onsite stormwater management could significantly reduce revenues for a local stormwater management program. Each community should evaluate whether charges and credits proposed for its utility are likely to promote onsite stormwater management and whether mechanisms are in place to ensure that onsite stormwater management achieves the desired environmental results.

Finally, public acceptability and political support is important to establishing a utility rate structure, whether or not it includes a credit approach. The nature of local government is that key players in utility design and implementation are seldom the key players in local politics. In designing a credit approach, a utility can attempt to minimize controversy by developing education and involvement programs for informing and gaining the support of local government officials and the public.

Case Study of Issues Associated with Proposed Credits in Indianapolis, Indiana

The City of Indianapolis is currently attempting to design a credit approach for its proposed stormwater utility. Considerable controversy has arisen over the proposed utility and a credit system is under consideration in part to help overcome general opposition to new charges or taxes. Through a credit system, utility planners and local elected officials are attempting to make the proposed stormwater utility charges more equitable and acceptable politically. The credit system in the most recent draft ordinance (Proposal No. 657, 1997) is a relatively complex approach to provide a reduction in stormwater user fees for nonresidential properties based on 1) certain qualifying conditions (location in relation to a major waterway), 2) activities that mitigate the impact of increased stormwater runoff from a property on a continuing basis, or 3) activities that reduce the city's cost of providing stormwater management ser-

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vices to a property. The draft ordinance outlines a two-tiered credit that is based on watershed area as well as the size of the onsite detention/retention basin. The city will also develop a proposed Storm Water Credit Manual for use in reviewing and acting upon applications for credit. A credit application fee is also authorized in the draft ordinance. Efforts to establish a credit system for onsite detention/retention have addressed concerns of property owners and generally increased the perceived fairness of the proposed rate structure, and it is clear that the proposed utility could not be adopted without some type of credits. Inclusion of a credit system, however, has not been sufficient to ensure adoption of the stormwater utility and overcome other objections.

Conclusion

Economists have long advocated pollution charges as an approach to achieve greater flexibility and efficiency in pollution control. If such charges are set to reflect the environmental damage actually caused by polluted discharges, economic theory suggests they can create incentives for each user to choose the least-cost option—paying a pollution charge or implementing pollution control requirements. Making credits available on stormwater utility charges for implementation of onsite stormwater management can create comparable incentives for users and potential efficiency gains by lowering the total costs of a stormwater management program. Additional research is needed to evaluate the efficiency and equity issues associated with credits and stormwater utility charges. Until the economic and data issues in establishing a credit approach are better understood, communities considering a credit approach should examine the experience of those utilities that have implemented credits to evaluate whether such approaches are appropriate for local stormwater management goals and problems.

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Conservation Design for Stormwater Management

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Background

The State of Delaware has developed a manual to provide guidance for site design which incorporates conservation into land development (DDNREC and BC, 1997). The intent is to provide an incentive for land developers to retain and incorporate existing natural site features into the site development process and thereby reduce or eliminate the need for structural stormwater management controls. Other benefits are certainly realized through Conservation Design, such as more closely approximating the predevelopment water budget, protection of habitat, and reduced overall impact to the receiving system. Site features discussed in the manual include:

- Wetlands
- Floodplains
- Forested areas
- Meadows
- Riparian buffers
- Soils
- Other natural features

Design procedures are provided which allow site designers to incorporate practices inherently known to be good, but which have not had the detailed design guidance that ensures plan approval. That guidance is provided in the manual for a variety of situations. The design approach is flexible enough to allow for various conservation practices to be combined on one site and to quantify the benefits of that combination.

It must be emphasised that structural controls will still be essential on many sites. A heavily wooded site having a significant portion of the tree canopy removed will still have a significant increase in stormwater runoff, even with aggressive conservation planning. The practices detailed in the manual are provided as additional tools in the stormwater management toolbox. They supplement structural control practices and may, in some situations, eliminate or reduce the need for structural practices while providing attractive site amenities.

Limitations of Structural Stormwater Management

Most stormwater management programs place a heavy reliance on implementation of structural stormwater management facilities. These facilities include ponds, both wet and dry; infiltration; filtration; and other variations of them all. The implementation of these facilities is necessary for their water quantity and water quality benefits and is expected to remain integral to program implementation, but there should not be an overreliance on them. These practices, in and of themselves, cannot eliminate adverse impacts of urban development. In addition, there are a number of limitations to structural facilities.

A stormwater management program relying solely on structural practices has a number of weaknesses.The existence of these weaknesses has been recognized for some time, but there has been little information available on alternative approaches that would justify their inclusion in a stormwater management program. The following items present some of the weaknesses.

- Lack of flexibility in site design
- Altered site hydrology
- Expense
- Loss of site area
- Potential increased impacts to site and watershed natural resources
- Configuration of development
- Connection of impervious areas
- Disregard of site resource conservation benefits
- Maintenance obligations

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Conservation Design Approaches

Conservation Design approaches reflect a totally different philosophy towards site design which integrates stormwater management into the very core of site design, as opposed to an afterthought. These approaches can include an almost endless universe of practices, strategies, planning, and common sense. The manual doesn't include all potential components, but provides guidance and information on many that are currently recognized where data exists or can be generated to substantiate their benefits from a water budget perspective.

It is important to develop a conservation ethic which treats stormwater runoff as a "resource" rather than a "byproduct" of development. As such, there are a number of key site design components to consider:

- Reducing impervious surfaces
- Constructing biofiltration practices
- Creating natural areas
- Leaving areas undisturbed
- Clustering development

Conservation approaches are discussed throughout the manual, but some are briefly discussed here to provide an initial awareness of the range of options that will be discussed later in greater detail. Examples of conservation approaches include the following.

Reducing Impervious Surfaces

Impervious surfaces (roads, roofs, sidewalks) prevent the passage of water through the surface into the ground. Water must then be transported across the surface to a point of discharge. Reducing the total amount of imperviousness is the single most important conservation tool available. Residential subdivisions can reduce the width of roadways, or design the roadways to limit the total length needed to service individual properties. Roof downdrains should not be directly connected to streets when providing splash blocks, but should discharge the water away from impervious surfaces (sidewalks, streets) to allow for a greater amount of water to infiltrate into the ground.

Just as important in limiting impervious surfaces and separating roof drains from direct connection to streets is the need for education of homeowners regarding their responsibility to ensure continued function of these practices. Homeowners often change the orientation of downspouts or otherwise redirect lot drainage to impervious surfaces which undoes a lot of conservation benefits. Community education and involvement is integral to effective program implementation.

Constructing Biofiltration Practices

The use of vegetative swales and buffer strips can provide a significant water quality benefit in addition to reducing the total volume of stormwater runoff. The primary processes involved in their performance are filtering of pollutants contained in stormwater runoff, and infiltration of runoff into the ground.

Even where curbs are needed to restrain traffic movement to paved surfaces, curb cuts or openings can be placed to allow water to pass off of the paved surface into a biofiltration facility. This would allow for both public works and stormwater objectives to be attained.

Creating Natural Areas

In many site development situations, the predevelopment condition may be farmfield or other disturbed condition. Creation of a meadow as open space would have significant stormwater management benefits for both water quantity and water quality. The area, if well designed and constructed, could become an attractive amenity to a community and enhance the value of the properties.

Leaving Areas Undisturbed

Many sites have existing resources which, in addition to other values, have stormwater management benefits. These natural systems include forested areas, wetlands, and other areas of natural value such as meadows.

Forested areas provide for rainfall interception by leaf canopy. In addition, an organic "duff area" develops on the woodland floor which acts very much as a sponge to capture the water and prevent overland flow. In addition, trees use and store nutrients for long periods of time. Trees also moderate temperatures during the summer and provide wildlife habitat, thus providing other environmental benefits.

Wetlands are valuable resources and provide numerous benefits including flood control, low streamflow augmentation, erosion control, water quality, and habitat. They are very productive ecosystems whose maintenance would have significant water quantity and quality benefits. Where they exist on a land development site, they could become an important element in site design.

Cluster Development

How a site is developed and to what degree the entire site is utilized will have a significant impact on stormwater runoff from the site. Conventional land development encourages sprawl, while innovative approaches to land development can provide significant stormwater benefits. Cluster development encourages smaller lots on a portion of a site, allowing the same site density, but leaving more site area in open space. Clustering designs residential neighborhoods more compactly, with smaller lots for narrower single-family homes, found in traditional villages and small towns. Cluster development can provide for protection of site natural areas, while at the same time reducing total site imperviousness by reducing the areal extent of roads.

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The Conservation Design Procedure in Detail

Conservation Design can be thought of as a series of questions which must be asked as it is applied to each site. If site designers rigorously address all of these questions, the Conservation Design procedure will have been accomplished, and the "answers" will be successfully identified for each site. The overriding objective is to achieve a new way of thinking about site design.

The procedure has been kept simple by intention. It is grounded in effective and complete site analysis, and an upfront commitment by the site designers to inventory and evaluate the various "systems" which define the site and which pose problems as well as opportunities for site development. The more clever the development "tinkering" can be, the more successful Conservation Design can become. Extra effort up front pays important dividends in the long run. Conservation Design requires a major departure from the conventional mindset of stormwater disposal - which is a reactive end-of-the-line process forcibly imposed onto a development program. Conservation Design is proactive in the best sense of the word, based on understanding natural system opportunities which enable us to integrate essential stormwater quality and quantity management objectives into the devlopment design from the very beginning.

Rather than provide a lengthy discussion of conservation design procedures, this paper provides a checklist of items or "questions" which should be considered in conservation design. Those questions are listed as follows:

1. Site Analysis Background Factors: How do Background Site Factors Affect the Conservation Design Process?

   Hydrologic issues:
   - Is the site tidally dominated?
   - Does the site flow to special waterbodies with special water quality needs?
   - Are there known downstream flooding problems?
   - The site is located in what watershed?
   - Does the site discharge into 1st, 2nd, 3rd order streams?
   - Is the site in the upper, middle, or lower part of the watershed?

2. Site Analysis Site Factors Inventory: What Site Physical Factors Affect Conservation Design?

   Site size and shape:
   - Does site size limit Conservation Design?
   - Does site shape or other factors limit Conserva tion Design?

   Natural features:
   - What is the basic site hydrology?
     - Perennial streams?
     - Intermittent swales?
   - Describe site soils
   - Describe site vegetation
   - Describe site critical features:
     - Do wetlands exist?
     - Are there floodplains?
     - Are there riparian areas?
     - Are natural drainageways present which are not perennial streams per se?
     - Are there special habitat areas?
     - Do special geological formations exist (i.e., carbonate)?
     - Do steep slopes exist?
     - Are there high water table, bedrock, other limitations?

   Built/developed features:
   - Does the site have centralized sewer?
   - Does the site have centralized water?

3. Site Factors Analysis: What Site Factors are Constraints and Opportunities in terms of Conservation Design?

   Site Constraints:
   - Where should building and roads be avoided?
     - In terms of vegetation?
     - In terms of soils?
   - Are any areas off limits for all forms of disturbance?

   Site Opportunities:
   - Where does most recharge occur?
     - In terms of vegetation?
     - In terms of soils?

4. Building Program: How do Building Program Factors Enter into the Conservation Design Procedure?

   Can the proposed building program be reduced in terms of total number of units?
   Can the <u>type</u> of units be modified (e.g., from single-family to townhouse)?
   What is existing site zoning?
   Are zoning options allowed?
   Have building setbacks been made to be flexible?
   Have innovative development concepts such as zero lot line or clustering been considered?
   What does the comprehensive plan indicate for the site and adjacent areas?
   What are the adjacent land uses?
   Other Management/Regulatory issues:
   - What municipal/county requirements exist for stormwater?
   - Will some aspects of Conservation Design require waivers?
   - What other municipal/county requirements exist for land development?
   - Will some aspects of Conservation Design require waivers?

5. Lot Configuration and Design: How Can Lot Configuration and Overall Site Design Prevent Stormwater Generation?

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Have lots been reduced in size to the maximum degree?
Have lots/uses been clustered/concentrated to the maximum degree?
Have lots been configured to avoid critical areas?
Have lots been configured to take advantage of effective Conservation Design mitigative practices?

6. Impervious Coverage: Have Impervious Surfaces Been Reduced as Much as Possible?

   Have road lengths and widths been reduced to the maximum degree?
   Have driveway widths and lengths been minimized to the maximum degree?
   Have parking ratios and parking sizes been reduced to the maximum extent?
   Has potential for shared parking been examined fully?
   Have cul-de-sacs and turnarounds been designed to minimize imperviousness?
   Have sidewalks been designed for single-side movement?
   Can porous surfaces be used for overflow parking, low impact shoulders, other applications?

7. Minimum Disturbance/Maintenance: Has Disturbance of Site Vegetation and Soils Been Minimized?

   Has maximum total site area, including both soil and vegetation, been protected from clearing and any other type of development disturbance?
   Are zones of open space maximized?
   Do these open space zones make sense internally, externally?
   In terms of individual lots, has maximum lot area, including both soil and vegetation, been protected from clearing and other development-related disturbance?
   Do structures correspond to site features such as slope, in terms of type of structure, placement on lot, elevation, and so forth?
   Have revegetation opportunities been maximized throughout the site?
   Have revegetation opportunities been maximized in critical areas such as riparian buffer zones?

8. Use of Mitigative Conservation Design Practices: Which Practices are Most Effective and How Can Their Positive Effects be Maximized?

   Are vegetated swales with check dams being used?
   Are vegetated filter strips with level spreading devices being used?
   Are berms and other terraforming technique being used in conjunction with zones of natural vegetation?

9. The Conceptual Stormwater Management Plan: How Can All Preventive Approaches and Mitigative Techniques be Integrated into an Optimal Conservation Design Plan?

   How has the stormwater plan been integrated into the overall site design?
   Has prevention been maximized through Conservation Design Approaches?
   Has mitigation been maximized through Conservation Design Practices?
   What other benefits are achieved through Conservation Design (i.e., open space, enhanced marketability, cost reduction, habitat protection, stream water temperature, biota impacts, other stream impacts?)

10. Stormwater Calculations: How Has Conservation Design Affected Stormwater Calculations? What Conventional Stormwater Techniques are Necessary to Manage Any Residual Stormwater Need not Mitigated by Conservation Design?

    How has impervious cover been reduced?
    What are the implications for Curve Numbers?
    How have total runoff volumes been affected?
    Has time of concentration been maximized?
    How has peak discharge rate been affected?
    How has recharge volume been affected?

11. Selection of Additional Stormwater Controls: If Conservation Design has not Fully Met all Stormwater Requirements, What Additional Requirements Must be Provided?

Watershed Wide Approaches

While not a focus of the manual, watershed-wide considerations are important and should be the context from which many resource-based land development decisions are made. The manual strongly supports watershed-based approaches to land use decisions. This context is important from a number of perspectives.

- Watershed approaches allow for a recognition and consideration of where growth distribution should occur.
- Consideration of land use from a watershed perspective allows for a greater awareness of the cumulative impacts of watershed development.Impervious surfaces are important to consider if downstream areas are to be protected.
- A comprehensive approach to resource protection can be developed and implemented based on consideration of watershed specific issues such as steep slopes, high water table, the need for aquifer recharge, etc.
- A watershed approach allows for developers and the general public to understand the basis by which land use decisions were made in a rational format which can be easily understood.
- Land use decisions based on watershed-wide analyses provide the local government with a basis for making land use decisions that can be defended.

As desirable as watershed-wide approaches are, it must be recognized that significant resources and costs may be

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needed to accomplish those efforts. Depending on the goals of the effort, significant data needs may exist.

Conclusion

Over the past 20 years, stormwater management has evolved from water quantity control, to water quality control, to attempting to address stream ecology. What has become apparent is that traditional end-of-pipe controls such as ponds do not provide the level of protection necessary to protect in-stream resources. We have gone full circle in again having to consider water quantity, but this time not just to reduce downstream flooding concerns. The total volume of water running off the land, in addition to riparian buffer protection, becomes critically important.

Site features, as mentioned in the abstract, must be considered integral to site development. Too often we have totally reconstructed a landscape for an individual's economic benefit only. We must recognize the economic and resource impacts that occur downstream from sites being developed.

There are ways to develop sites and protect or enhance existing resource values. It is not rocket science. If we as a society consider conversion of land to urban use as a desirable societal product, we must do more than accept the adverse impacts that those site activities cause. We can minimize adverse downstream impacts if greater weight is given to existing site resources. In many situations, as shown in case studies, land can be developed, less expensively using greater protection of existing site resources, than when using a conventional approach.

The choice is ours.

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Results of the Site Planning Roundtable

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The Site Planning Roundtable, originally convened by the Center for Watershed Protection in October 1996, brought together representatives from various national planning organizations, development and environmental communities, and local government. The goal was to provide the technical, professional, and real-world validation required to promote environmentally sensitive, locally relevant, and economically viable development. In line with this goal, the Roundtable has developed a set of 22 Model Development Principles that aid local planners and zoning officials in identifying how existing ordinances can be modified to reduce impervious cover, provide effective stormwater treatment, and conserve natural areas. These principles are not national design standards. Instead, they identify areas where existing subdivision codes can be changed to better protect streams, lakes and wetlands at the local level.

Conventional zoning standards outline minimum lot areas, setbacks, frontages, and road widths, often resulting in significant impervious cover in the form of wide streets, expansive parking lots, and large-lot subdivisions. Planners, landscape architects, and developers can utilize a wide range of innovative site planning techniques to reduce imperviousness at the site level. In some cases, full utilization of these techniques requires changes to outdated zoning regulations or inflexible subdivision codes. The Model Development Principles focus on changing these regulations and codes. Each principle presents a simplified design objective; techniques for achieving the objective should be based on local conditions.

Residential Streets

An important objective of the Site Planning Roundtable effort was to identify practical and cost-effective strategies to overcome barriers to implementation of the Model Development Principles. One particular area of concern emerged: residential road width. Most local governments model their residential street design standards upon state and/or federal highway criteria, although the traffic capacity and function of residential streets differ considerably from that of highways. Consequently, residential street widths tend to be wide rather than narrow. Efforts to reduce road widths are often met with strong opposition on a variety of fronts. Local planners and engineers are reluctant to modify standards due to safety concerns. Public works officials wish to maintain adequate access for emergency, service, and maintenance officials. Residents voice concerns about impacts to parking.

The following discussion will present alternative design standards to reduce imperviousness and demonstrate how many of the impediments to narrow streets are already being overcome with careful site design.

Perceptions and Realities: Parking Demand

Why are residential streets wide? Parking is a major factor. On-street parking on both sides of the street can increase site imperviousness by approximately 25% (Sykes, 1989). Limiting parking to one side of the street or the use of queuing lanes can significantly reduce this imperviousness. The reduction of on-street parking is often cited as an impediment to narrow streets. This impediment can be overcome. In Portland, Oregon, parking is accommodated through the use of "queuing streets" which are 20' or 26' wide (Figure 1).

Perceptions and Realities: Safety

The potential for increased vehicle-pedestrian accidents is an often cited reason for prohibiting narrow streets. Many studies, however, indicate that narrow streets may actually be safer than wider streets. The Federal Highway Administration (1996) noted that narrow widths tend to reduce the speed at which drivers travel, providing greater driver reaction time. Further, in a study of over 5000 pedestrian and bicycle crashes, a narrow road was a factor in only two cases (FHWA, 1996). Unsafe driving speed, on the other hand, contributed to 225 accidents.

Case Study: Longmont, Colorado

The City of Longmont, Colorado, is experiencing rapid growth. The quality and type of new development has become an important issue as more development and non-conventional site designs are proposed. Part of this discussion involves acceptable residential street design. Swift and Associates (1998) examined over 20,000 police reports to determine the relationship between street design

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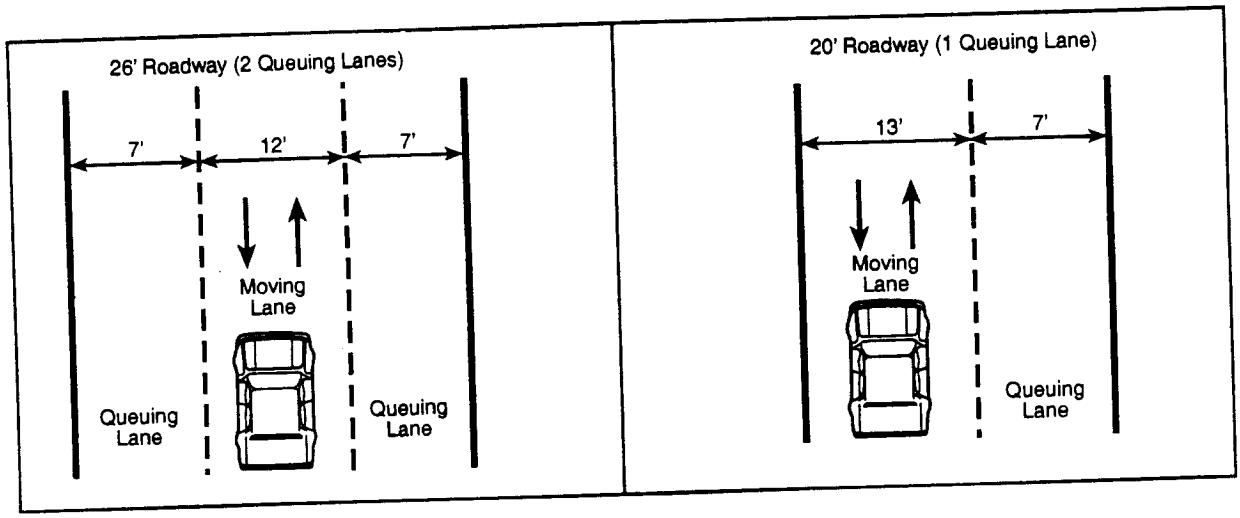

Figure 1. Queuing Lanes in Portland, Oregon.

and safety. The study focused specifically on residential streets with maximum average daily traffic (ADTs) of 2,500. Accidents attributable to poor road conditions or substance abuse were excluded from the study. The results of the Longmont study indicate that in general, narrow, curved streets can be safely used in residential developments. Specifically, streets between 22 to 30 feet in width were found to be the safest (see Figure 2).

Perceptions and Realities: Adequate Access

The conventional wisdom is that very wide streets are needed to provide adequate access for emergency, service and maintenance vehicles. But the facts do not support this concern:

- Trash trucks require only a 10.5' travel lane (Waste Management of Montgomery Count, 1997), with a standard truck width of approximately 9' (BFI of Montgomery County, 1997).
- Half-ton mail trucks, smaller than many privately owned vehicles, are generally used in residential neighborhoods. Hand delivery of mail is also an option (US Post Office, 1997).
- School buses are typically nine feet wide from mirror to mirror. Many jurisdictions require only a 12' driving lane for bus access.
- Snowplows, mounted on pick-up trucks, with 8' width, are common. Some companies manufacture alternative plows on small "Bobcat" type machines (Frink America, Incorporated 1997).
- A number of local fire codes permit roadway widths as narrow as 18' (Table 1).

Narrow Streets

Reduced pavement widths can significantly reduce the impervious impact of residential developments. Site designers should consult with public works, emergency service, and residents to confirm that the community's needs are met. Adequate access, parking, and safety can be ensured through careful site design.

In addition to environmental benefits, significant construction cost savings can be achieved by building narrower streets. Pavement construction costs are approximately $15 per square yard. Suppose, for example, that a local jurisdiction currently requires all residential streets with one parking lane to be a minimum of 28 feet wide. The jurisdiction then adopts a new standard: 18 feet wide queuing streets. This new standard would reduce the overall imperviousness associated with a 300-foot road by 35% and construction costs by $5,000. Additional economic benefits include reduced clearing and grading costs and reduced long-term pavement maintenance costs.

Acceptance of the narrow streets design requires implementation as a flexible, locally adapted strategy. Therefore, the Model Development Principles must be consistent with the larger community goals (both economic and environmental) that are put forth in comprehensive growth management, resource protection, and watershed management plans. Finally, the Site Planning Roundtable encourages local, state, and federal agencies to provide the technical support, financial incentive, and regulatory flexibility needed to promote and implement the Model Development Principles, and to fundamentally change the way development takes place.

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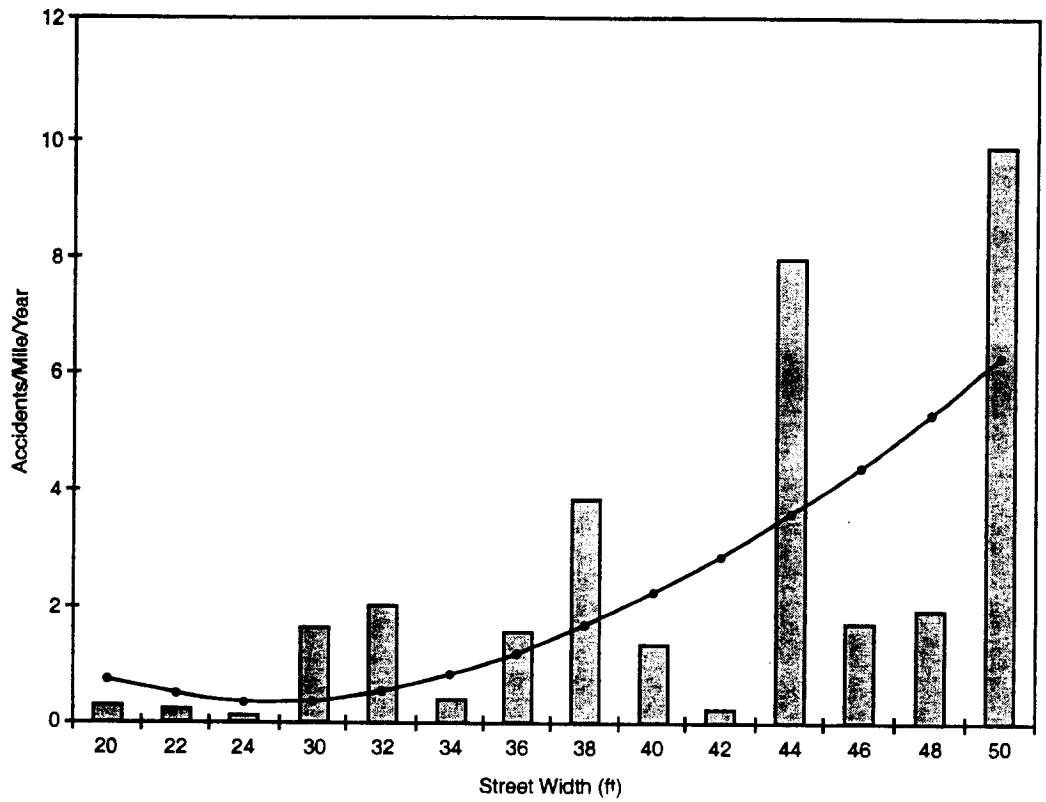

Figure 2. Relationship between street width and accidents in Longmont, Colorado, based on Swift and Associates (1998).

Table 1. Street Width Requirements for Fire Vehicles

Width	Source	Comments
18-20'	US Fire Administration (Cochran,1997)	Represents typical "fire lane" width
24' (on-street parking) 16' (no on-street parking)	Baltimore (MD) County Fire	Road width
18' minimum	Virginia State Fire Marshal	Road width
24' (no parking) 30' (parking on one side) 36' (parking on both sides)	Prince Georges County (MD) Department of Environmental Resources	Road width
20'	Prince Georges County (MD) Fire Department	Road width

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Retrofitting Conservation Designs into the Developed Landscapes of Northeastern Illinois

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Abstract

There is a small but growing trend to retrofit developed landscapes in northeastern Illinois with more environmentally friendly designs. The motivation for these activities varies from site to site. Commonly, those initiating retrofits are hoping to reduce landscape maintenance costs, fix erosion problems, improve water quality, enhance aesthetic conditions, and/or reduce flooding.

Four types of retrofitting have been identified. Retrofitting conventional turf landscapes with native prairie/wildflower vegetation is one of the more visible and exciting trends. Notably, prairie landscaping has become a desirable option for several high-visibility corporate, school, and government campuses. Stream channel retrofitting also has become common. Many of the recent stream projects have been stimulated by demonstration projects, funded through Section 319 of the Clean Water Act, to reduce bank erosion. Detention retrofitting has been done on a more limited scale. Detention basin retrofitting has been initiated to improve stormwater runoff control, reduce shoreline erosion, improve aesthetics, and limit excessive Canada goose populations. Conversion of storm sewers to open drainageways has been performed on a very limited basis to improve treatment of runoff pollutants.

While water quality has not been the principal impetus for many of the recent retrofit projects, the water quality benefits can be substantial. As a consequence, watershed managers are increasingly recommending large-scale retrofitting to enhance the beneficial uses of urban and suburban waterbodies and are selling the retrofit concept on aesthetic and cost-saving grounds.

Motivation for Retrofitting

From a water resources perspective, there is a growing realization that the developed landscape of northeastern Illinois has some serious design and performance flaws. One of the most obvious reminders of this fact is the frequent and increasing incidence of damaging floods. Where there was once considerable ignorance regarding the causes of flooding, the average resident can now readily relate local and regional flooding to new roads, parking lots, and subdivisions and their contributions of increasing stormwater runoff.

Similarly, but to a lesser degree, there is an increasing awareness that conventional urban development designs have led to impaired water quality and degraded recreational uses of waterbodies. Contributing factors include polluted stormwater runoff and the outright destruction of wetlands and riparian corridors.

Increasingly, concerns also are being raised about the sustainability of traditional development. In particular, individuals are questioning the costs of maintaining both the structures and the landscapes that dominate the developments of the recent past. In particular, questions are being raised about the continued reliance on turf grass as the dominant landscaping material for commercial, office, and residential developments. There are concerns about the expense and environmental impacts of a maintenance approach that relies on frequent mowing, irrigation, and the extensive use of chemicals for fertilization and pest control.

Fortunately, there is also a growing awareness and appreciation of an alternative urban design ethic that incorporates "natural" elements into developed properties. This alternative ethic is based on both ecologic and aesthetic considerations. Central to this ethic is the belief that it is both possible and desirable to commingle natural areas and materials with developed landscapes. Elements of this ethic include use of native plants for landscaping, preserving or restoring natural buffers at the edges of developments, and incorporating native ecosystems — prairies, wetlands, and woodlands — into development site designs.

Cumulatively, this growing awareness of the shortcomings of conventional design and the potential benefits of naturalistic approaches provide the basis and motivation for retrofitting elements of the developed landscape.

Types of Retrofitting

Retrofitting, as described in this paper, includes a range of activities. In other contexts, some of these activities might

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be termed rehabilitation, restoration, or renovation. Regardless of terminology, retrofitting is assumed to involve a substantive, long-term change to an existing facility or landscape resulting in demonstrable improvements to water quality and aquatic ecosystems.

Four types of retrofitting projects have been implemented in northeastern Illinois and are documented in this paper. They are:

- converting conventional turf landscapes to native vegetation,
- restoring eroding and/or channelized stream and rivers,
- retrofitting stormwater detention basins, and
- converting, or "daylighting," storm sewers to open drainageways.

Converting Turf to Natural Landscapes

For decades, exotic turf grass has been the dominant landscaping material for almost all new development in the region. Considering that the pre-development landscape on most sites is cropland, and considering the relative paucity of remnant native landscapes such as prairies, savannas, and woodlands, this landscaping philosophy is not surprising. Recently, however, there has been growing and enthusiastic support for *natural landscaping*, an alternative approach that utilizes native plants that are adapted to the local climate and soil (Northeastern Illinois Planning Commission, 1997).

Natural landscaping applies to an array of landscaping techniques that incorporate native vegetation, particularly prairie, wetland, and woodland plants. Natural landscaping also includes natural drainage techniques, such as swales and vegetated filter strips, instead of storm sewers and artificial drainage channels.

Benefits

The benefits of natural landscaping are the most broad-ranging of any of the retrofitting techniques. In addition to water quality benefits, they include flood reduction, habitat enhancement, improved air quality (climatological benefits) aesthetic enhancement, and cost-savings.

Water quality benefits are derived in two ways. First, unlike conventional landscapes of turf grass and ornamental plants, native plants do not usually require chemical additives after their initial establishment. Fewer applications mean greatly reduced runoff of fertilizers, pesticides, and herbicides. Second, natural landscapes, particularly with the deep root zones of many native plants, can effectively soak up, filter, and transform contaminated stormwater runoff from roadways and parking lots, greatly reducing the pollutant loads discharged to the "receiving stream".

Flood reduction occurs due to the greater infiltration capacity provided by deep-rooted native plants. In contrast to the four-to-six-inch root zones of turf grass, the dense root systems of native prairie plants commonly extend several feet into the soil, creating passageways for the rapid infiltration of precipitation and runoff. The dense root systems also enhance evapotranspiration.

Habitat enhancement is provided by the diversity of plants found in a natural landscape, in contrast to the conventional near-monotypic stand of turf grass. Native wildflowers host numerous birds and insects whereas turf will not. Natural landscapes are particularly valuable adjacent to lakes and streams where they provide habitat for aquatic insects and amphibians that spend time both in the water and in terrestrial environments.

Improved air quality results, in part, from reduced usage of lawn maintenance equipment that discharges hydrocarbons and nitrogen oxides into the environment. Native plants also enhance air quality by filtering out particulates and converting carbon dioxide to oxygen. In a related manner, natural landscapes (particularly trees) provide climatological benefits via shading and wind breaks, thereby moderating temperatures and enhancing human comfort.

Natural landscaping also can provide *aesthetic enhancement*. Natural landscapes provide a great variety of textures, colors, and shapes that vary seasonally. They also attract a variety of wildlife, particularly birds and butterflies, enhancing their visual appeal.

The *cost-savings* in maintenance costs of natural landscapes can be dramatic. Natural landscapes need little of no fertilizer or pesticide, as already noted. They do not require regular irrigation, as does turf. They also need little or no mowing. The preferred long-term maintenance approach for many naturally landscaped sites is prescribed burning, performed every one-to-three years, much as it was done by Native Americans.

Local Examples

Natural landscaping has been retrofitted onto numerous sites throughout northeastern Illinois. Most of the retrofitting has occurred on residential lots. In terms of community impact, some of the most striking retrofits have been on large office campuses and public properties.

Residential landscape conversions commonly involve the replacement of turf grass or annual flowers with perennial wildflowers and prairie grasses. Most conversions occur gradually as residents discover and appreciate the advantages of natural landscaping. In some cases, virtually the entire lawn is converted, although in many cases significant buffers of conventional landscaping are retained to minimize potential conflicts with neighbors. As public education and acceptance increase many communities that formerly prohibited tall grasses and ungroomed landscapes, are now growing more flexible toward natural landscapes.

Commercial and office campus sites provide some of the most impressive examples of natural landscape con-

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versions owing to their high visibility and large expanses of land. Notable examples include the AT&T corporate campuses in suburban Lisle and Naperville and the Prairie Lakes commercial redevelopment in Homewood. Natural landscape conversions on these sites have been motivated by a combination of factors, including corporate image, and the influence of employees.

Public properties, notably schools, parks, government centers, and roadways, are increasingly popular targets for landscape retrofitting. Schools provide logical retrofit opportunities considering their typically large expanses of high-maintenance turf and the potential educational opportunities. Wheaton Warrenville South High School, for example, retrofitted 2.5 acres of turf into dry and wet prairie vegetation. Students have been involved in planning, planting, and management of the restored areas.

Restoring Stream and Rivers

The streams and rivers of northeastern Illinois reflect a history of abuse and neglect. Over 40% of the stream miles have been channelized or severely modified to provide agricultural drainage or accommodate urban development. Uncontrolled development of upstream watersheds has led to severe flooding, streambank erosion, and water quality degradation. Hence, there is a great need and opportunity for retrofitting. In the context of this paper, retrofitting is used to describe stabilization or limited rehabilitation of the physical characteristics of the channel and its riparian zone. Retrofitting includes stabilization of eroding banks, enhancement of instream habitat, and restoration of the near-stream riparian zone (Dreher 1998).

Benefits

Potential benefits include bank stabilization, improved water quality, improved habitat, and enhanced aesthetics.

Bank stabilization can be achieved by a number of techniques. The preferred approach would incorporate the use of soil bioengineering techniques that are largely based on natural materials and vegetation. Effective bank stabilization reduces the loss of riparian land, protects streamside infrastructure (such as bridges and buildings), and reduces sediment load.

Water quality improvement can be accomplished by a number of retrofit techniques. One way to improve water quality is to restore the natural pollutant filtering capability of the riparian zone and floodplain. This is most readily accomplished by re-planting streambanks and riparian buffers with native vegetation, particularly indigenous wetland, prairie, and woodland plants that were common prior to settlement. Another way to improve water quality is to stabilize stream temperatures. Often, degraded streams suffer from over-heated conditions during the summer due to a loss of shading in combination with overly-wide, shallow channels. Establishing native vegetation, particularly along sensitive headwater streams, can result in substantial improvements.

Stream habitat is improved by restoring critical elements such as meanders, pools, riffles, and natural substrate to a degraded channel. Some habitat improvements can be readily accomplished as part of other stream rehabilitation projects. For example, the addition of rock substrate at appropriate locations can accomplish both riffle enhancement and stream bed stabilization. However, restoration of meanders to a straightened stream channel typically would be performed as an independent restoration project.

Aesthetic enhancement is accomplished in most restoration projects, whether intended or not. Replanting of native vegetation to stabilize an eroded streambank, for example, also results in a visual improvement. Intentional enhancement of a degraded stream may be in the best interest of some land developers, particularly of residential properties, to improve the marketability of a project. For example, re-meandering and replanting a channelized stream can convert an ugly ditch into an attractive stream in the eyes of home buyers.

Local Examples

The Northeastern Illinois region has been the fortunate recipient of funding to implement several stream restoration demonstration projects. These projects, funded principally through Section 319 of the Clean Water Act, have provided highly visible models for others to emulate.

Two projects in which the Northeastern Illinois Planning Commission has been involved are restorations of the Skokie River and Flint Creek, located in suburban areas north and northwest of Chicago, respectively (Price, 1997). Both projects successfully demonstrated the use of "soil bioengineering" techniques for streambank stabilization and both restored significant areas of riparian buffer. The projects also attempted to restore instream aquatic habitat, although on a limited basis.

Several *parks and golf courses* have implemented stream restorations to beautify their grounds and to reduce the loss of recreational lands to excessive streambank erosion. Such restoration projects have utilized public education to overcome the historical bias that favors manicured landscapes over ungroomed "natural areas."

Several large *residential developers* have implemented, or initiated planning for, significant stream restoration projects. These have been done with the intention of enhancing the visual appeal of the developments and/or accommodating development on sites constrained by floodplain locations. Whatever the motivation, aquatic habitat, water quality, and hydrologic functions stand to benefit substantially. A notable example is the Fox Mill development in west suburban Kane County. This project resulted in the re-meandering of the ditched headwaters of Mill Creek and the conversion of a large riparian buffer to native wetland and prairie.

Citizen organizations have been active in stream restoration. These include loosely organized volunteer groups

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as well as land trusts that actually own and manage land. Restoration activities have ranged from streambank stabilization projects supported by volunteers to more-extensive restorations of riparian buffers and wetlands. Notable examples include the restoration of a 1000-foot buffer along Flint Creek by the Citizens for Conservation and the rebuilding of 200 feet of river edge and restoration of a riparian buffer along the Middle Fork of the North Branch Chicago River by the Lake Forest Open Lands Association (Price, 1997).

Retrofitting Stormwater Detention Basins

Northeastern Illinois communities have required stormwater detention for new development since 1970. Currently, the vast majority of municipalities and counties have detention ordinances. While local ordinances are some of the most restrictive in the nation with respect to flood prevention, most have not incorporated water quality designs until the recent past. For example, many older basins are simple dry bottom designs, often with paved low-flow channels, that provide little pollutant removal benefit. Thus, there are substantial opportunities to retrofit older basins to enhance their effectiveness. Retrofitting projects can range from simple repairs or alterations to major rehabilitation, depending on the project objectives and the existing conditions of the basin.

Benefits

Detention basin retrofitting can be targeted to a range of objectives. These include improved pollutant removal, improved flow control, reduced maintenance, and enhanced aesthetics.

Improved pollutant removal can be achieved in most older dry-bottom basins. One retrofitting technique is to revegetate basin bottoms with wetland plants in place of turf. If paved low-flow channels exist, they can be replaced with vegetated swales. Pollutant removal also can be enhanced by excavating settling basins at the inlets and/or outlet of the existing basin. Settling basins can greatly enhance the removal of suspended solids and attached pollutants. Outlet structures also can be modified to increase detention times for small-to-moderate-sized storms, thereby enhancing pollutant settling. Finally, pollutant removal can be enhanced in basins where inlets and outlets are located in close proximity, causing short-circuiting. This is accomplished by lengthening flow paths through the construction of low berms.

Improved flow control can be readily achieved in some basins by modifying the outlet structure. This is particularly beneficial in older basins that were designed to control only the 100-year discharge. The outlets of such basins can be retrofit with restrictor plates or berms, or replaced with completely new structures, to provide control of smaller storm flows, such as the 2-year event. Such control is important in stabilizing downstream flows to reduce the potential for streambank erosion. However, it should be recognized that increasing the control of smaller storms will result in less storage availability for larger flood events.

Reduced maintenance is an objective in many older basins. One way to reduce maintenance is to replace turf grass on basin bottoms and side slopes with low-maintenance native vegetation. Depending on wetness conditions, either wetland plants or prairie grasses and wildflowers can be used. Native vegetation requires only occasional mowing or prescribed burning. Maintenance needs can be reduced in some basins by excavating settling basins at basin inlets. Properly sized basins can concentrate the settling of most particulate matter at the inlets, thereby facilitating long-term sediment removal from the basin.

Enhanced aesthetics is a concern in many older basins. Problems range from eroding shorelines to excessive populations of Canada geese. Shoreline erosion in wet-bottom basins can be controlled with the introduction of buffers of water-tolerant native vegetation into shoreline zones. Where erosion is severe, installation of soil bioengineering measures, as previously described for stream restoration, can be effective. Introduction of shoreline buffers of taller native plants also can be an effective control for Canada geese. Indications are that geese are not comfortable moving through tall vegetation and are, therefore, more likely to seek conventionally landscaped basins. They also prefer short turf grass as a food source.

Local Examples

In contrast to natural landscape conversions and stream restorations, there has not been a widespread retrofitting of detention basins in northeastern Illinois. Perhaps the most likely explanation is that detention basin owners generally are unaware of the performance deficiencies of older basins, particularly their inability to effectively remove stormwater pollutants. Without regulatory incentives for such retrofitting, little has occurred. Detention basins owners are more responsive to maintenance and aesthetic concerns. Consequently, older detention basins are being retrofit with native vegetation and shoreline stabilization measures.

The most notable detention retrofitting project in the region is a demonstration project funded in part by the U.S. EPA through Section 319 of the Clean Water Act. This project involved an older dry-bottom basin in the Village of Flossmoor, approximately 30 miles south of Chicago (Price and Dreher, 1995). The basin had a failed outlet structure, due to sediment clogging, and a paved low-flow channel between its principal inlet and the outlet. Retrofitting involved the excavation of stilling basins at the two basin inlets, excavation of a permanent pool at the outlet, installation of a new multi-orifice outlet structure, and revegetation of the basin bottom and side slopes with native wetland and prairie vegetation. The retrofit basin provides substantially improved pollutant removal and improved hydraulic control of small storms, and requires substantially less maintenance than the former basin. Local residents have indicated their satisfaction with the aesthetics of the retrofit basin, as well.

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Converting, or "Daylighting," Storm Sewers

The term "daylighting" refers to the elimination of a storm sewer or culvert and its replacement with open channel flow. On the principle that open drainage systems provide certain natural and aesthetic functions not provided by artificial, underground systems.

Benefits

There are several potential benefits of converting closed pipes to open channels. These include improved water quality and hydrologic functions as well as aesthetic benefits. These benefits are optimized when the open channel is designed as a natural, unlined swale or stream.

Several *water quality benefits* are likely to result from storm sewer daylighting. By running stormwater through an open, vegetated channel, runoff pollutants can be filtered and transformed by a combination of physical and biological processes. These processes, similar to those occurring in natural swale and stream systems, are constrained in closed pipes by inadequate light and the absence of natural substrates.

Improved *hydrologic functions* also are likely to result from sewer daylighting. For example, flow in an open swale will have some opportunity for infiltration, thereby enhancing natural recharge and baseflow. Natural open channels also can better dissipate flow velocities, potentially reducing downstream flooding and channel erosion.

Aesthetic benefits are most readily appreciated at the point where a storm sewer or culvert discharges to a receiving stream or lake. Eliminating storm sewer bulkheads, in particular, is likely to enhance the visual appeal of a bank or shoreline, and enhance recreation in the waterbody (Dreher and Price, 1997).

Local Examples

There are few reported examples of storm sewer daylighting in northeastern Illinois, although there is increasing interest in the concept among watershed managers. One documented project is the conversion of several hundred feet of large diameter storm sewer serving downtown Barrington, northwest of Chicago. As part of a redevelopment project, the Village removed the storm sewer and replaced it with a meandering wetland swale. The objectives of the project were to enhance the quality of the discharge into nearby Flint Creek and to improve the appearance of the property from an adjacent park and a planned trail (Price, 1997).

Several area watershed groups are currently discussing opportunities for daylighting. In particular, the Friends of the Chicago River, as part of a comprehensive watershed management project, have identified storm sewer daylighting as a remedial best management practice (BMP). It is notable that in some areas of the North Branch Chicago River watershed, nearly all of the historical surface drainage system has been replaced by storm sewers. In this context, storm sewer retrofitting will not only benefit the river but also may educate local residents to the advantages of a natural drainage system looks like.

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Impacts of On-site Sewage Systems and Illicit Discharges on the Rouge River

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The focus of the Rouge River National Wet Weather Demonstration Project (Rouge Project) is to clean up the Rouge River in southeast Michigan. The Rouge River watershed includes portions of the City of Detroit and 47 communities west and northwest of Detroit. Water quality sampling and models show that eliminating combined sewer overflows (CSOs) alone will not ensure that water quality standards are met or that the river can be used for all purposes the public desires. The information presented in this paper shows that non-stormwater sources, such as on-site sewage systems and illicit discharges, are major contributors to the contamination of the river.

On-site Sewage Disposal Systems

On-site sewage disposal systems (OSDS) exist in urban areas of the Rouge River Watershed and are contributing to surface and groundwater pollution. In Wayne, Oakland, and Washtenaw Counties, OSDS requirements exist only for the installation of such systems; operation and maintenance are the responsibility of the owners. Rouge River OSDS failure rates documented in surveys conducted in 1994, 1995, and 1997, varied between 17 and 55%. The 1995 and 1997 studies evaluated 528 residential OSDS for failures, which were identified by the following:

- observation of sewage discharging from the area of the OSDS
- observation of liquid on the ground surface of the disposal field
- identification of a pipe draining sewage from the disposal field area
- heavy vegetation on or near the OSDS
- detection of dye in surface water downstream from the septic tank after dye was placed in the septic tank.

Some surface waters in sewered communities have been found to be unsafe for human contact due to high levels of *E. coli* bacteria. Sources of *E. coli* here include CSOs, sanitary sewer overflows and leaks, illicit connections, wildlife excrement, and failing OSDS. Illicit dumping of septic wastes from recreational vehicles may also contribute to occasionally high *E. coli* counts. Some surface waters that drain areas which are not served by sanitary sewers have also been found to be unsafe for human contact due to high *E. coli* bacteria counts. These unsewered areas are served by OSDS. Other potential sources of *E. coli* bacteria here are illicit discharges through pipes that drain to surface water, wildlife excrement, and agricultural operations.

In order to perform the OSDS surveys, it was necessary to identify the locations of systems installed in the study areas. While local health departments issue permits for OSDS installations, three of the four health departments in the Rouge River Watershed have not entered permit information into computerized databases. Local communities did not have records of OSDS. Each of the surveys required the development of a database of OSDS permit information to help locate systems in the field. Census data from 1990, which included information about the numbers of OSDS by city block, were used to identify areas served by OSDS. Information available from water utility billings from each community was also included in the OSDS databases, along with the results from the field surveys. Results from compiling the databases were surprising to local governments. These included the following:

- The 1990 census data showed that there are more than 1,700 OSDS within Detroit city limits. City officials were surprised at these figures, since on-site sewage disposal systems are illegal in Detroit.
- Further checking indicated that there are areas in the City not served by sewers. Although a City policy ex-

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ists requiring payment for sewer connections, no follow-up has occurred to verify that connections have taken place.

- Utility billings were screened for three communities in the study area to identify households that were paying for water but not for sewer service. Homeowners who were billed for sewer service (e.g., were connected to sewers) were not surveyed. Results through September 1997, revealed the following:

Community	Total Number of Homes Contacted	Homes with City Sewer Connection	Total Homes with OSDS	Homes with OSDS that are Failing
A	152	18	134	28
B	239	22	217	45
C	53	15	38	6

- Of the 444 homes contacted, 55 reported that they are connected to the city sewer but are not paying sewer charges.
- Of the 389 homes surveyed with OSDS, 79 had failing systems (20% failure rate).
- None of the communities were aware that homes were connected to sewer systems but not being billed for service. These communities are losing revenue by not recovering costs of operation and maintenance of the sewer system from these customers.

Wayne County Study

The Rouge Project Office funded a grant to Wayne County to conduct visual surveys of OSDS for homes located along a Rouge River tributary that drains into an area being considered for canoeing. Because of high *E. coli* bacteria counts, canoeing on the river has been discouraged. Through October 31, 1997, the County had conducted surveys for 427 homes to identify signs of OSDS problems. Of these, 90 systems have been described as failing or potentially failing—a failure rate of 21%. Typical descriptions from the field notes were as follows:

- Sewage backup in the home.
- Gray water discharging to the ground surface.
- Standing water on top of the gravel seepage field.
- Mushy area, associated with the back end of an apparent seepage field.
- Illicit connection and undersized septic tank (100 gallons) drained by a trench type (long single perforated pipe) seepage field.
- Black sludge residue and toilet paper debris around surface of the septic tank covering.
- Growth of cattails, wet marsh on the face of a downward sloping hill.

Oakland County Study

Another study, which took place in the Oakland County portion of the Rouge River Watershed, included dye testing septic tanks and stream sampling for fecal coliform, *E. coli* bacteria, and benthic macroinvertebrates. Study results are as follows:

- Of 49 surface water sampling sites, 43% had a daily geometric mean for *E. coli* bacteria of 1,000 or more per 100 milliliter of sample.
- The macroinvertebrate study was done to determine the water quality of streams in the survey area. A scale was developed to rate macroinvertebrate and water quality. The results in the study area ranged from 7, which indicates poor water quality, to 20, which is considered good water quality.
- Dye testing conducted in 1994 showed that 53% of the homes tested had discharges to the river.
- An optical brightener test to detect laundry waste was conducted at the river sites where dye was collected. These were all negative.
- Dye testing conducted in 1995 showed a 39% failure rate for OSDS in the communities surveyed.

Future Direction

The future direction of this effort is to establish, in cooperation with local health departments, an on-site sewage management program in each community. Communities are also encouraged to address on-site sewage systems in applications for general stormwater permits issued by the State of Michigan under the National Pollutant Discharge Elimination System (NPDES) Program. Septage disposal problems are being addressed with septage haulers and disposal facilities.

Costs

Grant expenditures for the Wayne County and Oakland County surveys were $105,000 and $61,000, respectively. This includes amounts spent by agencies to administer the grants, conduct the investigations, and complete necessary reports and other documentation. Additional costs were realized by communities required to extend sewers to problem areas and homeowners who were required to correct failing OSDS or connect to available sewers.

Illicit Connections

From 1987 through 1996, Wayne County investigated approximately 3,340 businesses and industries for illicit connections to the storm sewer system. Approximately 9% of the facilities inspected were found to have illicit connections. The elimination of these improper discharges has diverted raw sewage and other pollutants from the river to the wastewater treatment plant. Findings of the investigation are as follows:

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- An average of 2.6 improper connections were found at businesses that had illicit connections.

- The majority of illicit connections in non-residential facilities were drains connected to storm sewers. These included floor drains, trench drains, interior catch basins, oil separators, machine process water drains, and sump pumps. The categories of illicit connections found were floor drains (46%), sinks (20%), washing machines (15%), toilets (11%), and a variety of others (8%).

- A method to prioritize the investigation was developed based on the Standard Industrial Classification (SIC) of businesses. The prioritization method was successful for locating illicit connections. It was not helpful in locating illicit discharges of *E. coli.*

- The use of aerial, infrared, and thermal photography to locate discharges that have a higher temperature than that of the stream, or locations where algae might be concentrated, is in the experimental phase. The aerial infrared experiment also examines soil temperatures, land surface moisture, and vegetative growth. Assumptions are that (1) a failing OSDS will have increased moisture in the surface soil, (2) the area will be warmer, and (3) vegetation will grow faster than the surrounding area. These differences should be visible in the digital data. Analysis of data collected has been hampered by a lack of resources to conduct field work needed to develop computer references.

- To date, there have been no definite correlations among field tests for ammonia, anionic surfactants (detergents), and *E. coli.*

- Field crews and members of the public have identified a significant number of improper discharges to the river through visual observations.

- Stable isotopes of oxygen and hydrogen have been used to determine the presence of sanitary sewer water in discharges.

- Visual observations and liquid flow testing indicate that 160 manholes and outfalls have suspicious discharges.

- Based on these findings, the estimated number of potential illicit violations in the entire Rouge River Watershed is 5,260.

It is estimated that 51 million gallons of liquid will be discharged from illicit connections within the Rouge River Watershed. Field work performed during dry weather (72 hours without precipitation) identified 160 manholes and outfalls that had ammonia readings of 1.0 or greater, or had visible conditions that were cause for further investigation. All of these manholes were investigated for ammonia, anionic surfactants, and *E. coli.* The bacteria results showed that 16 locations had *E. coli* bacteria counts greater than 5,000 per 100 ml. These locations will have a detailed investigation to locate the source. Many of the areas with suspicious discharges are residential areas. Municipalities will be requested to participate in finding improper connections.

Work performed in 1997-98 will focus on locating sources of *E. coli* that are impacting the Rouge River from Nankin Dam to Merriman Road, a distance of approximately 1.5 miles. This is a prime area for recreational activity, which would be significantly increased if the river was safer for human contact. The work will begin at the 16 manholes/outfalls with high *E. coli* bacteria counts mentioned above. Manholes located upstream of the sampling sites will first be tested. Each highly suspect source will be dye tested to confirm whether or not it is contributing to high *E. coli* counts. Sampling of Tonquish Creek and its tributaries that drain into the proposed canoeing site will be conducted during dry weather to determine if the same process will be needed in this area.

An example of going upstream from a trouble spot to locate the source of pollution is the investigation of a storm sewer relief drain. An 11-foot storm sewer had been under suspicion for several years. In 1997, samples were collected from manholes located upstream from the discharge point. Samples collected in June from one of the laterals connected to the sewer had *E. coli* counts of 8,160 and 9,600 per 100 ml. Additional samples were taken five days later. Levels of *E. coli* from samples taken progressively upstream in the storm sewer where the 9,600 per 100 ml count was found were 12,560; 24,000; 160,000; and 9,600 per 100 ml. A lateral of this sewer had a result of 4,800 *E. coli* per 100 ml. The manhole with the 160,000 per 100 ml count was found to be the "hot spot."

The results of this sampling activity were shared with city officials who decided to have the sewer televised. However, the tapes did not show any suspicious connections. Plans were then made to begin dye testing at homes located next to the storm sewer. Before beginning the process of dye testing, another sample was taken at the trouble spot to have current information. The results of that sample indicated less than 8 *E. coli* bacteria per 100 ml. Following discussion with the city, it was agreed that dye testing would be postponed. It was also agreed that residents would be informed of the sampling activities that had taken place on their street. At this point, it was felt that the high *E. coli* count from the initial sample may have been due to an incident of someone dumping wastes directly into the sewer. A letter was sent to residents asking them to let the city or county know if they had knowledge of any practices that could have resulted in the high *E. coli* counts. As a result of these sampling activities, this investigation, and community interest, sampling continues on a monthly basis on this street.

Future areas to be checked will be identified based on citizen complaints, a review of manhole and outfall sampling to determine contributing conveyances, and instream/insewer sampling to localize the area. Using Rouge Project GIS, maps have been prepared for tracking the sampling of manholes and outfalls. These maps and the sampling

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data help municipalities identify and prioritize areas that need to be further investigated.

Future Direction

The future direction of illicit connections/discharges is to have each community in the Rouge River Watershed commit to actively exploring illicit connections/discharges. Grants and assistance from county agencies are available to communities and agencies. As part of an application for a General Stormwater Permit from the State of Michigan under the NPDES program, a community is required to develop an Illicit Discharge Elimination Plan. The Rouge Project assists communities in preparing these applications. Elements of the Illicit Discharge Elimination Plan recommended to be included are (1) a legal basis for the program, (2) how problem areas will be identified, (3) how the sources will be pinpointed, and (4) how to achieve correction, evaluation, and reporting.

Costs

The budget for the Illicit Detection Investigations Program in 1996-97 was $735,000. The budget for the 1997-98 program is $599,000. Besides field work, this budget includes trials of different methods of investigation, testing, and subcontracting for special studies. There is also a significant cost for grant administration. Not included are costs likely to be incurred by businesses to correct illicit connections or the cost to communities of televising sewers to locate illicit connections. The 1997-98 program provides nine full-time-equivalent employees. Of these, six perform investigations and water sampling.

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Stormwater Management in an Environmentally-Sensitive Urban Bushland in Sydney, Australia

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Introduction

This paper outlines and discusses the challenges faced, and the techniques employed, by the Upper Parramatta River Catchment Trust in developing and executing an integrated stormwater management strategy in an environmentally sensitive bushland reserve in the middle of the Sydney metropolitan area, in New South Wales, Australia.

The key element of the strategy was a 30-metre high concrete wall and associated structures, which form a large flood detention basin in the bushland reserve. Completed in mid 1996, the flood basin protects over 300 properties in several residential and commercial areas from major floods.

As part of the overall project, complementary measures such as bushland regeneration, creek bank stabilisation, water quality monitoring and sediment and litter traps were implemented in the reserve to overcome serious existing degradation of the bushland and protect it from further degradation.

The adopted strategy was considered to be the best possible compromise between the need to protect the bushland environment and the need to protect homes, offices and factories from flooding. This paper briefly outlines the history of the project and some of its more noteworthy features.

Background

The Upper Parramatta River watershed or catchment forms the headwaters of Sydney Harbour in the city of Sydney, in the State of New South Wales, Australia. As shown in Figure 1, the catchment is located in the centre of the Sydney metropolitan area, between 20 and 30 kilometres west of the Sydney central business district. The outlet of the upper catchment is at a weir separating the freshwater and estuarine sections of the river. This is located just downstream of the Parramatta central business district, the main commercial centre for western Sydney.

The area of the catchment is 110 square kilometres and has a population of 230,000. Most of the catchment's 70,000 properties are single detached residences, although there are extensive commercial and industrial areas and an increasing number of multiple occupancy dwellings. Much of the catchment is urbanised, although there are significant areas of remnant vegetation in bushland reserves and urban forests, most located along the creeks.

Figure 2 shows that the two main tributaries of the Parramatta River are Toongabbie Creek which drains the west and south of the catchment and Darling Mills Creek which drains the northeast.

The upper Parramatta River catchment includes portions of the areas of four local authorities (called local councils in Australia). Baulkham Hills, Blacktown and Holroyd cover the upslope areas in the catchment's north, west and south respectively, whilst Parramatta covers the catchment floor.

Although the catchment has experienced flooding since the earliest days of European settlement from 1788, the problem was compounded by rapid urban development of the catchment in the 1960s and 1970s. At that time the hydrologic impacts of urbanisation were not appreciated and there was a lack of cooperation among the local councils. The growing flood threat only became apparent during a series of storms in the late 1980s, which inundated hundreds of properties many times. Detailed flood studies showed that, in storms only marginally larger than those experienced, substantial areas including much of the Parramatta central business district would be flooded.

The historic Lennox Bridge over the Parramatta River at Parramatta increases the risk of serious flooding in the Parramatta central business district. Constructed in 1837, this sandstone arch bridge is the third oldest bridge in Australia. Unfortunately, its arched waterway opening means that, as the river level rises, less and less additional waterway is available to pass flows.

Hydraulic studies showed that once the river level reaches the top of the arch opening, as almost occurred in the 1986 and 1988 floods, the river would break its banks and quickly flood substantial areas of the central business district. The flood risk, and the consequential development

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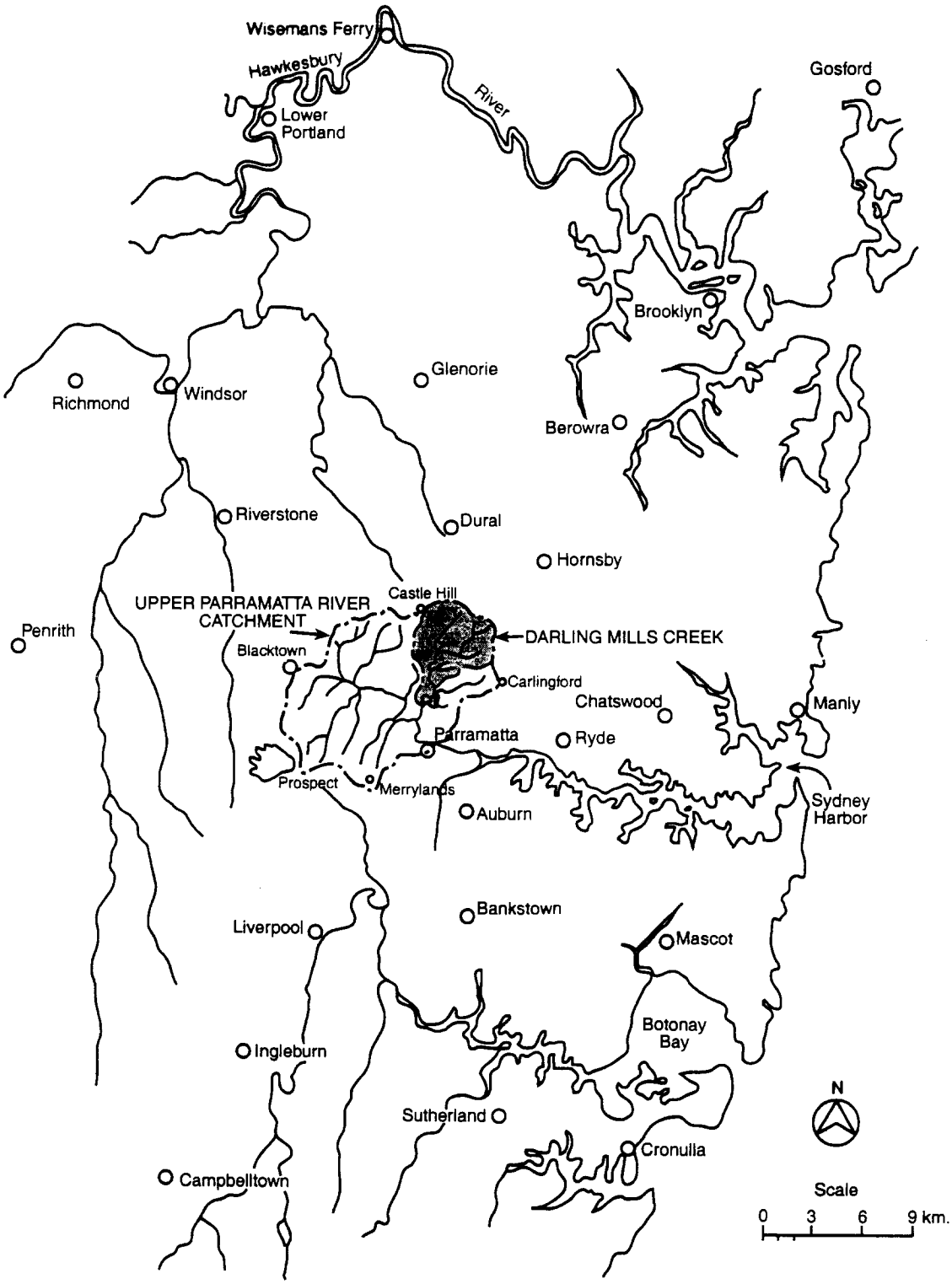

Figure 1. Location of catchment in relation to Sydney

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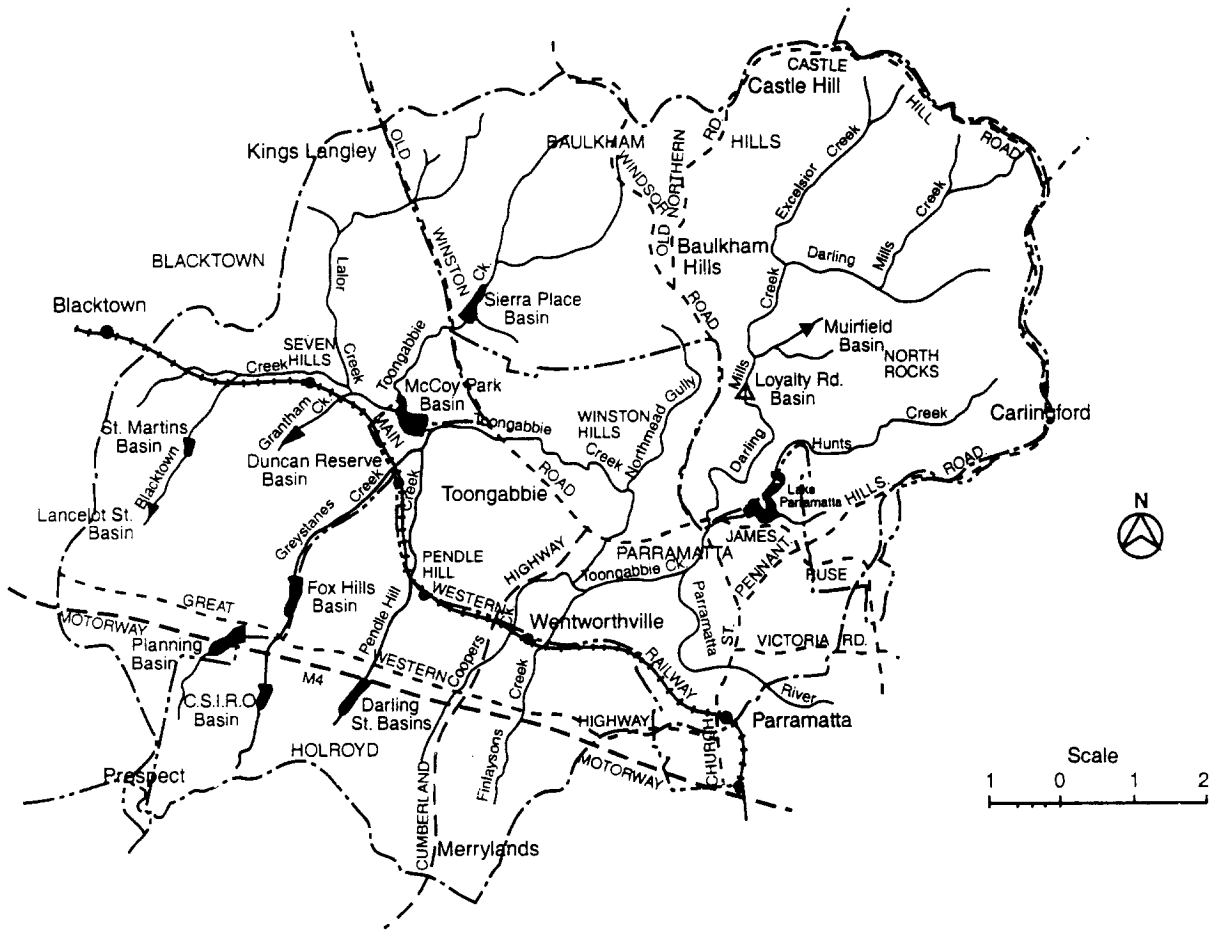

Figure 2. Location of existing flood detention basins in catchment

restrictions imposed by the Parramatta Council, caused the affected parts of the central business district to become rundown.

A proposal in 1991 to reduce peak flood levels by demolishing Lennox Bridge was narrowly voted out by Parramatta City Council. Soon after, the state government put a 'permanent conservation order' on the bridge, effectively ruling out that option.

Attention then turned to alternative solutions, particularly in the Darling Mills Creek sub-catchment. Although this sub-catchment occupies only one-third of the upper catchment, it contributes half the storm flows because of its higher rainfalls and steeper slopes. Moreover, as shown in Figure 2 there were seven large flood detention basins and several smaller basins in the Toongabbie Creek sub-catchment; there were none along Darling Mills Creek.

For most of its length, Darling Mills Creek flows through a heavily vegetated and steep-side sandstone valley up to 100 metres deep contained within a publicly owned bushland reserve, Excelsior Reserve. Urban development surrounds Excelsior Reserve on all sides. Despite this, the dense vegetation and the deep valley, together with massive sandstone cliffs, create a sense of wilderness which many local residents make use of for bushwalking and quiet relaxation.

The extensive urban development surrounding Excelsior Reserve has also caused the creek's water quality to deteriorate and the bushland either side of Darling Mills Creek to become degraded. The reserve was infested with weeds, there was severe creek bank erosion in places, and considerable litter and nutrients were swept into the reserve by stormwater. Despite voluntary bushland regeneration work by local residents, Baulkham Hills Shire Council has been unable to allocate the funds needed to overcome the degradation.

Investigations

Because the issues involved two local council areas Baulkham Hills and Parramatta, investigations into a possible solution were sponsored and managed by the Upper Parramatta River Catchment Trust, the catchment management authority for this area.

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The trust had been established in 1989 following representations by local authorities and the public in the wake of several major floods. It is a state agency funded by an annual levy on all catchment properties. The trust's charter is to coordinate flood mitigation, water quality and related catchment management activities in accordance with the NSW Government's integrated catchment management policy. It has a part-time board of twelve directors (nominated by the four local councils and relevant state government agencies) and a permanent staff of seven.

The future of the trust is currently being considered in the context of a planned catchment management body for the entire Sydney Harbour catchment.

Initial studies by the trust showed that a large flood detention basin in the bushland reserve on Darling Mills Creek was the only viable way to protect from floods over 300 properties in the Parramatta central business district and four other flood-liable areas along Darling Mills Creek and the Parramatta River.

This immediately posed three significant challenges because it meant:

- building a large structure in an environmentally sensitive bushland;
- storing floodwater temporarily in a heavily vegetated reserve (until then detention basins had only been built on grassed playing fields); and
- creating a large detention basin in the upstream local council area (Baulkham Hills) to protect properties in the downstream local council area (Parramatta) from flooding.

Community Consultation — Support and Opposition

From the outset, a community group formed to protect the bushland reserve signalled its strong opposition to the proposal for a large flood basin in the reserve. Despite its strident opposition, the group participated in the workshops and meetings held whilst the proposal was being formulated and its impacts assessed. Other groups from flood-liable areas further downstream were vocal in expressing support for the flood basin.

A project steering committee was formed with representatives of the trust, both councils, government agencies and groups supporting and opposed to the flood basin. The committee met regularly during the course of the investigations. In addition, two community workshops were conducted to obtain community feedback at critical stages. Finally, a two-day Value Management Workshop to assess the most favourable alternative was conducted by an independent facilitator. Progress reports were published in the trust's quarterly newsletter delivered to all catchment households.

Investigations and Design

Although not legally required, the trust decided to prepare an environmental impact statement (EIS) to support a formal application to Baulkham Hills Shire Council to implement the stormwater strategy, including the large detention basin. This ultimately involved some 40 separate environmental, social, economic and technical studies over two years at a total cost approaching $500,000 (Australian dollars). The issues addressed ranged from Aboriginal archaeology, acoustics and air quality, to water quality, weed control and zoology.

Of these issues, the most controversial was the likely impact of the basin on vegetation in the impoundment area. It was claimed that the raised flood levels inside the basin would spread weeds to higher elevations in the bushland reserve, whilst raised soil moisture levels would eventually kill off mature trees. This issue proved difficult to resolve because of the apparent absence of other detention basins in bushland areas. In general, studies of other issues found that impacts of the basin would be minor and could be effectively mitigated.

A key part of any EIS is the assessment of all feasible alternatives to the favoured option. Opponents of the large basin sought to frustrate the EIS by repeatedly proposing alternatives that, it was claimed, would avoid the need for the basin. Each had to be examined carefully. In all, 20 alternatives or groups of alternatives were assessed, including different large basins, groups of small basins, bridge modifications, a flood tunnel by passing Parramatta, acquisition of flood-liable properties and/or compensation.

The first stage of the evaluation confirmed that only a large basin in Excelsior Reserve would protect all at-risk communities at an affordable cost. The second stage evaluation showed that the basin site near Loyalty Road, North Rocks, was clearly superior on environmental, social and financial criteria. This conclusion was confirmed at the Value Management Workshop run by an independent facilitator.

Different types of basin walls (dams) were also examined. On technical, environmental and financial criteria, it was found that a mass concrete wall constructed with roller-compacted concrete (RCC) would be best. The concept design was refined and detailed plans prepared. The design team included Ernest Schrader of the US, the world's foremost expert in the roller-compacted concrete technique. Mr. Schrader visited for a week during the design work, and again during the construction.

The basin wall was to be 23 metres (at the spillway) to 30 metres (at the abutments) high, and 110 metres long at its maximum height. It would comprise 23,000 cubic metres of RCC, with pre-cast concrete panels on its external faces. Its upstream face would be vertical. Its downstream face would consist of a series of steps and have an overall slope of 0.8 to 1. A central spillway, with walls either side, was designed and model tested to safely pass even the largest

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possible flood. A 2.5 by 2.7-metre culvert allows passage through the wall and contains a 1-metre square low-flow channel.

Some key issues and concerns identified during preparation of the EIS are listed in Table 1, together with how each was addressed.

The basin would have a maximum flood storage capacity of 1.5 million cubic metres. Detailed hydrologic and hydraulic studies conducted by the trust showed that the flood basin would reduce the peak flow in the critical 1 in 100 (1%) annual exceedance probability (AEP) storm by 75%, reducing the number of flood-liable properties further downstream from 313 to 75. However, all detained floodwater would drain away within 3 to 6 hours of the rainfall easing; and flows in the creek would be unaffected for more than 99% of the time.

Development Consent

In early September 1994 the trust submitted its development application to Baulkham Hills Shire Council. This was supported by an EIS comprising a 350-page main re-

Table 1. Darling Mills Creek Stormwater Management Strategy - Addressing Key Concerns

Issue or Concern	How Addressed
Most flood mitigation benefits in another local council area (Parramatta).	Flood basin 'packaged' with various environmental measures to address serious degradation of the Excelsior Reserve bushland and the creek.
Fear that temporary flooding of bushland would kill or degrade vegetation.	Flood studies showed that all stored water will drain away within 3 to 6 hours of heavy rainfall easing. Baseline surveys made of vegetation and creek channel against which future changes can be assessed. Similar detention basin in Adelaide Hills of South Australia found to have caused no significant harm to upstream bushland after 30 years.
Visual intrusion of large man-made structure into natural setting.	Basin wall sited within a creek meander so that the wall is only visible within 50 metres upstream and downstream. Texture, width and colour of external panels designed to blend in with shadows from nearby tall trees.
Loss of bushland area due to structure.	Basin wall constructed using roller-compacted concrete to minimize its 'footprint' — only 2 hectares in 300-hectare reserve.
Raised water levels in basin will spread weeds to higher levels in valley.	All weeds removed from the basin area and up slope to prevent the spread of weeds. Ongoing bush maintenance to control weed regrowth.
Basin wall will block use of track along bank of creek by hikers and animals.	Culvert through the bottom of the wall allows normal creek flows to flow through, and enables hikers and animals to pass from one side of the wall to the other.
High-velocity water discharging through culvert in basin wall (up to 15 metres per second) will scour downstream creek banks.	Dissipater structure and stilling basin designed and model tested to control the high-velocity flow out of the culvert under flood conditions and reduce its velocity before the floodwaters discharge into the downstream creek.
Construction truck movements up and down narrow unsealed track to site will cause unacceptable noise, dust and erosion.	A Rotec conveyor system was imported from the USA to deliver RCC and conventional concrete from a temporary batching plant near the reserve edge to the basin wall. The conveyor zigzagged its way between the trees, avoiding the need to remove any large trees.
Construction noise will disturb residents living in nearby houses.	Vehicle access to the construction site was through an adjoining industrial area. Construction was limited to daylight hours five and half days per week. A 24-hour per day 'hot line' operated by a specialist consultant received and dealt with all inquiries and complaints.
Need to construct basin wall quickly to minimize risk of flooding of construction works by rises in creek.	Precast concrete panels acted as formwork for placement of the RCC in layers and as the permanent external facing of the basin wall.
Concern about safety of basin wall in an extreme flood.	Basin wall has a central spillway, with training walls on either side, capable of passing the probable maximum flood. Steps on downstream face help dissipate energy of overtopping floodwaters. Design was approved by state agency responsible for dam safety.
Significant areas of bushland will be cleared to provide for stockpiles, storage and batching.	Areas able to be cleared strictly limited by contract with stiff penalties for non-compliance. RCC aggregate blended off-site. Materials delivered to site only when required.
Hikers may be trapped in basin by quickly rising floodwaters.	Studies showed rate of rise of floodwaters increased, but not unduly hazardous. Creek-bank walking track upgraded. Bypass walking track constructed up and around basin wall. New footbridge built over creek. Signs erected indicating egress routes.

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port and three volumes of specialist working papers, each of about 300 pages.

Because of strong opposition to the project from the small, but determined group, and the possibility that its decision could be appealed in the courts, city council was careful to allow everyone to have a say. The proposal was publicly exhibited and comments on the proposal and the EIS were invited. Letters of objection were received from 25 households and letters of support from 19. Council held a public meeting to discuss the proposal. About 60 people attended — equal numbers for and against the proposal. Council also had a retired judge conduct a mediation conference at which groups supporting and opposing the project put their case; but no compromise could be found.

At about this time the trust was advised of a similar large flood detention basin in the Adelaide Hills of South Australia, which temporarily inundated natural bush. An inspection by council members and senior staff showed healthy mature trees growing in areas subject to regular inundation and no significant infestation by weeds.

Finally, after seven months of comment and deliberations, the trust's application was approved unanimously by the council, subject to over 100 conditions previously agreed upon. Because government funding was required to help finance the project, the EIS was also submitted to the Commonwealth Government and, in due course, approved.

Construction

To avoid needless delay in commencing construction, detailed design plans had been prepared whilst the EIS was being finalised, before development consent was granted.

Within a month of development consent, detailed design plans were completed and tenders for construction called. Within three months, a $6 million contract to construct the basins wall was awarded and a consultant appointed to supervise the construction. The other environmental and structural measures, which formed part of the overall Stormwater Management Strategy, were carried out under separate contracts or by direct trust supervision of contractors. Construction was subject to severe environmental conditions reflecting the environmental sensitivity of the site, all monitored closely by the relevant agencies.

The main difficulty experienced during the 12-month construction period was frequent wet weather for four months. This caused the exposed excavation to be flooded 20 times, requiring exhaustive cleanup after each flood event.

Some of the more successful construction features were the pre-blended aggregate, the RCC mix, the conveyor system used to deliver RCC to the basin wall and the precast panels used both as formwork and the permanent facing of the wall.

Upon completion of the basin wall in July 1996, all disturbed areas were restored using previously salvaged plants, mulch, topsoil and rocks.

Other measures carried out as part of the overall project included:

- an alternative walking track around the basin wall site, incorporating a new timber bridge over the creek, a set of steps down a sandstone rock face and a formal viewing area;
- detailed vegetation transect and creek cross section surveys against which future changes can be assessed;
- removal of all weeds from the 10-hectare basin impoundment area and regeneration with suitable native plants, then maintenance for at least five years;
- extensive creek bank stabilisation to allow four-wheel drive vehicle access to the basin wall for maintenance;
- testing of water quality upstream and downstream of the basin wall site before and during construction;
- construction of a CDS (continuous deflective separation) pollutant trap and sediment traps on gullies leading into the reserve; and
- survey, trial excavation and ongoing monitoring of several rock shelters with Aboriginal archaeological potential within the basin impoundment area.

Conclusions

The largest flood detention basin in New South Wales, Australia, has been constructed in a degraded, environmentally sensitive bushland reserve a few kilometres north of the Parramatta central business district in western Sydney. The project not only addressed flooding, but also deteriorating water quality and bushland degradation. The local council approved it after exhaustive studies showed that a large basin was the only feasible way to cost-effectively achieve the flood mitigation objectives and that any adverse impacts on the bushland could be avoided or minimised. The environmental assessment undertaken was undoubtedly the most comprehensive ever carried out for a stormwater project in New South Wales.

The project illustrates the changing nature of urban stormwater control projects: the comprehensive investigation of all possible impacts, the detailed evaluation of alternatives, the community involvement, its multi-objectives, the strict environmental controls and the use of new construction techniques to minimise environmental harm. The increasing requirements for such projects mean that, in future, their proponents will have to accept the considerable challenges involved, and allow the necessary time frame and budget.

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Can a Steel Plant be Clean?

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Introduction

This paper discusses stormwater management, and in particular stormwater quality control, from a heavy industrial site located on the upper reaches of the Manukau Harbour, in Auckland, New Zealand.

Founded in 1962, Pacific Steel Ltd., a business unit of Fletcher Challenge Steel, is New Zealand's second largest steel manufacturing plant, and its largest recycling operation. The plant processes 200,000 tonnes of scrap steel including some 60,000 car bodies annually to produce a range of wire rod and reinforcing steel bar products. The Pacific Steel site covers some 20 ha (45 acres) and includes both steel manufacturing and scrap metal recovery operations. Prior to development, the site was under pasture.

The site is located on the southern shores of the Mangere Inlet, in the upper reaches of the Manukau Harbour (Figure 1). The outer Mangere Inlet is recognised as important for marine vegetation, and as a high tide roost for thousands of international migratory and New Zealand endemic wading birds, including a number of threatened species.

Significant improvements in the quality of Pacific Steel's stormwater discharge have been achieved over the past 5 years following the construction of a stormwater pond/wet

Figure 1. Map location

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and treatment device, and the implementation of a comprehensive stormwater management plan for the site.

The stormwater treatment device is used as a demonstration site for stormwater treatment from a heavy industrial area in the Auckland Region, and consequently has been extensively monitored by both Pacific Steel Ltd. and the Auckland Regional Council (ARC). This paper outlines the results of this monitoring. The paper also highlights how the site's stormwater management practices have improved, particularly in response to the results of this monitoring. Finally the paper discusses the practical experiences gained in the operation of a stormwater treatment device within a heavy industrial site.

New Zealand's Statutory Environmental Framework

By way of background, New Zealand's environmental statutory framework is set out in the Resource Management Act 1991 (RMA). This is an omnibus piece of legislation having consolidated some 56 pieces of legislation relating to the environment. The purpose of the RMA is to *"promote the sustainable management of natural and physical resources."* The RMA also sets out the powers, duties, and functions of the various authorities responsible for implementation of the Act.

The ARC is the environmental protection agency for the Auckland Region. With respect to stormwater management, the ARC has responsibilities for minimising natural hazards such as flooding, and for stormwater quality issues.

The RMA is an effects-based piece of legislation, and requires the effects, both positive and negative, of any proposed activity to be identified prior to commencement of that activity. Once identified, the RMA requires that any adverse effects are, as far as possible, avoided, remedied, or mitigated.

While focusing on the effects of a given activity, the RMA also enables a "best practical option" (BPO) approach to be taken to the discharge of contaminants to air, water, or land, or an emission of noise. This BPO approach is defined as the best method for preventing or minimising the adverse effects on the environment having regard to, amongst other things:

1. the nature of the discharge or emission and the sensitivity of the receiving environment to adverse effects
2. the financial implications on people and society
3. the effect on the environment of that option when compared to other options
4. the current state of technical knowledge and the likelihood that the option can be successfully applied

ARC Stormwater Management Program

The ARC initiated its stormwater quality control programme in the late 1980s, in response to growing concerns over the impact contaminated urban runoff was having on both the freshwater and marine receiving environments.

Auckland is characterised by the fact that it sits astride two major harbours. Its freshwater catchments are generally short and steep, with most discharging to low energy estuarine and upper harbour areas. Any contaminants that wash off the land, therefore, are rapidly deposited in these low-energy marine environments, and accumulate with time.

Given the variability of stormwater quality through time and from different land uses, the ARC adopted a "best practical option" approach to its management and, in particular, the treatment of stormwater.

At an early stage in the development of the stormwater quality control programme, the ARC prepared design guidelines for stormwater treatment devices (ARC 1992). Through this design manual, guideline removal efficiencies for stormwater treatment of 75% for suspended solids are promulgated. The ARC also requires all new development or redevelopment to address stormwater quality on a case by case basis (ARC 1995).

The ARC has established, in conjunction with a number of interested parties, a range of representative stormwater treatment demonstration sites, from which to monitor the effectiveness of a range of treatment devices under Auckland conditions. In addition to monitoring their effectiveness, the ARC has used these devices to undertake research to further characterise and quantify stormwater related impacts in the Region. One such site has been the Pacific Steel pond/wetland described in this paper.

Pacific Steel Ltd. - Stormwater Management

Historically, stormwater from the site was collected and discharged, largely untreated, via three stormwater outlets (Figure 2).

Following detailed investigations in the late 1980s, a stormwater treatment pond was commissioned in 1992, designed to treat runoff from the entire site up to a 1-in-5-year duration storm flow. The pond is located in the south-western corner of the site, and discharges via a single outfall to the "southern inlet," a small tidal arm of Mangere Inlet (Figure 3). The retrofitting of the stormwater treatment pond, described in more detail below, was accepted at the time by the ARC as the best practical option for stormwater quality control for the site.

In addition to the construction of the treatment pond, Pacific Steel Ltd. has introduced a number of day-to-day site management measures under the auspices of a stormwater management plan. The plan is designed to manage the site's stormwater system and minimise the initial contamination at source. It establishes protocols and sets frequencies for a range of site practices, such as regular storm drain inlet cleaning, street sweeping and dust suppression, installation of waste handling facilities,

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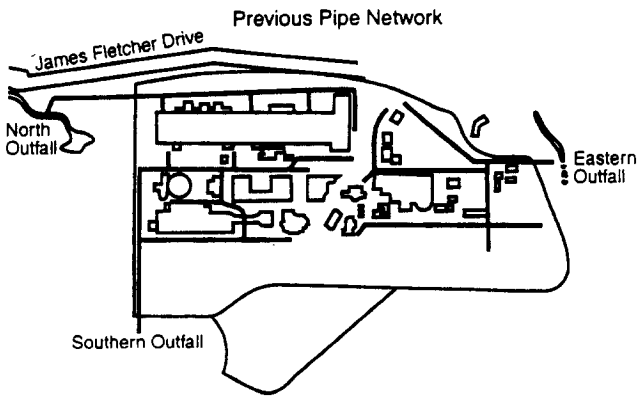

Figure 2. Previous stormwater network.

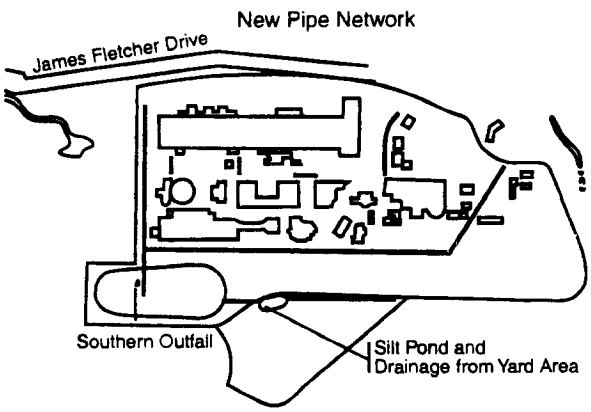

Figure 3. Current stormwater network.

wastewater audits, staff training, and emergency spill response plans. The implementation of the stormwater treatment and the management plan has dramatically increased the awareness of workers on site to stormwater related issues.

Sources of Contamination

The principal types of contamination of concern include: sediment, oils and grease, and a variety of heavy metals. Of the 20 ha site, some 45% is covered by paving, 20% by roofs, and the remaining 35% by stone covered storage yards.

Given the nature and activities on site, sources of potential stormwater contamination are varied and include:

1. Runoff from roads and roofs
2. Steel scrap stockpiles
3. Steel making waste stockpiles
4. Car shredder waste
5. Steel manufacturing (dust and fumes)
6. Truck washes

Typical contaminant concentrations in the site's runoff are shown in Table 1.

The variation in stormwater quality inflow to the pond, as outlined in Table 1, stems largely from the site conditions and the prevailing weather conditions. The 1990 results characterise stormwater runoff from the southwestern part of the site, which did not include runoff from the dirtier scrap recovery areas. Similarly the 1992 result characterised stormwater from two-thirds of the site, again without some of the scrap recovery area contribution. The 1994-1996 results represent the average stormwater quality as determined by monthly grab samples over that period. This sampling would have encountered a range of climatic conditions and is considered to represent the longer term average inflow concentrations for the site. As with the 1992 results, the 1997 results are the average inflow concentrations from sampling of specific storm events.

The variability of contaminant inflows and the influence of individual storm events is highlighted. A more detailed analysis of the long term monthly data has suggested that the inflow contaminant concentrations are reducing with time (ARC unpublished data). This is principally attributed to the improved site management practices carried out under the Stormwater Management Plan. The performance of the pond itself is discussed below.

Treatment Pond Design Characteristics

The treatment pond incorporates a permanent pond and a constructed wetland, as well as an oil trap and an emergency overflow (Figure 4). The pond was designed in accordance with standard stormwater management practices. The pond is some 200m long and has an overall volume of some 4,750 m^3, with an additional live storage of approximately 4,200 m^3. This volume is in excess of that required to meet ARC's guideline of 75% suspended solids removal (i.e., approximately 4,450 m^3). Total cost of the stormwater pond and site upgrade was in the order of $NZ 1M in 1992 ($US 0.6M).

The treatment device was initially commissioned in a staged manner as the re-routing of the site's drainage system took place. It was intended that only stormwater and emergency cooling water overflows pass through the pond. However, recent work related to PCB contamination in the outflow of the pond has identified other waste streams, such as truck wash effluent, which is also discharged to the pond. The impact of these additional waste streams on overall pond performance is currently being investigated.

Operational Experience With Treatment Pond

Experience to date has shown the pond to be effective at removing suspended sediments and other contaminants in the site's runoff. The pond is trapping an average of 3

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Table 1. Typical Pacific Steel Ltd. Stormwater Contaminant Concentrations, as Measured at the Inlet to the Stormwater Treatment Pond through time (g/m³).

Parameter	1990[1]	1992[2]	1994-1996[3]	1997[4]
Suspended solids	19	101.1	77	210
Total Oil and Grease	11	-	35	-
Copper (total)	0.018	0.14	0.14	0.48
Zinc (total)	0.18	1.6	2.94	7.4
Zinc (soluble)	0.09	0.2	0.37	0.23

1 Average stormwater quality prior to treatment pond from "cleaner" part of the site (Bioresearches, 1990)
2 Average inflow over six storm events to new pond prior to full diversion of entire site flows (Leersnyder, 1993)
3 Average ARC monthly grab sampling 1994-1996 (ARC unpublished data)
4 Average inflow concentration over four storm events in 1997 (NIWA, 1997)

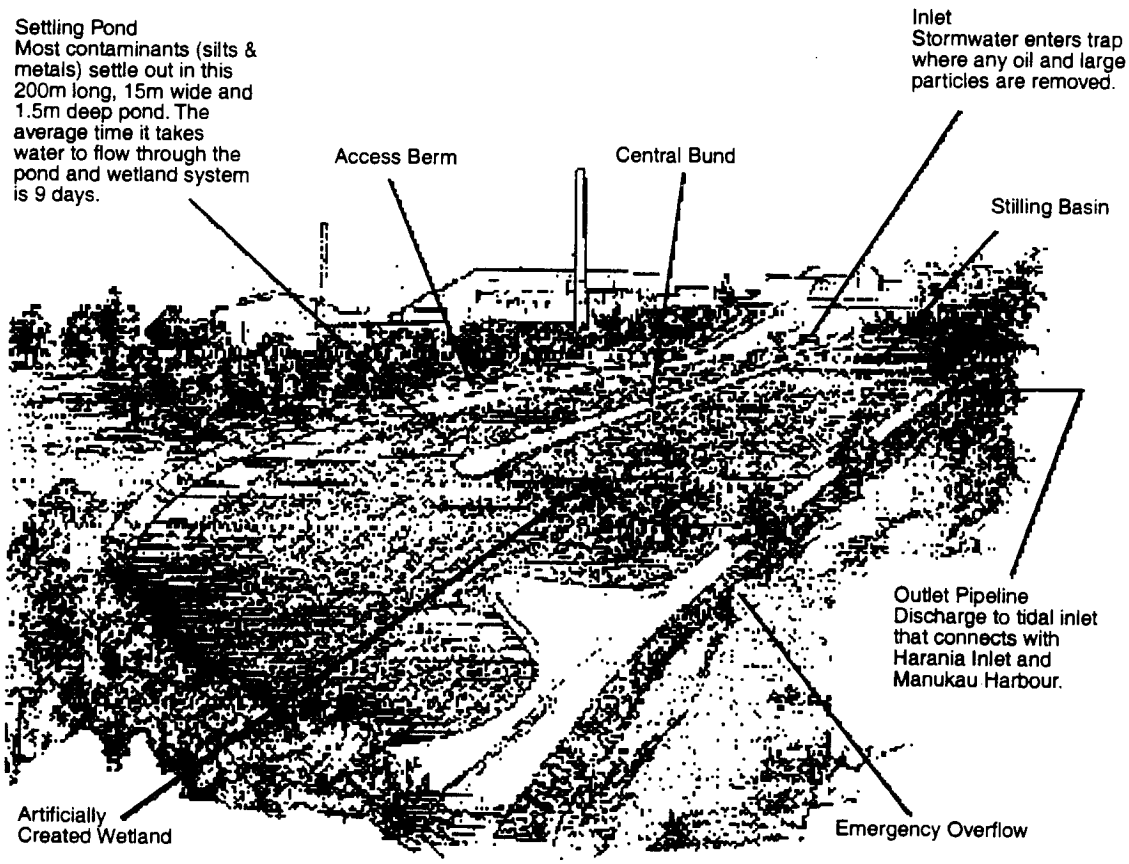

Figure 4. Pacific Steel stormwater treatment pond

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tonnes per week of suspended solids which otherwise would have been discharged to the wider receiving environment. In addition, as much of the site is covered, the site's storm drain inlets also trap significant quantities of material which is removed on a regular basis. A more thorough assessment of the pond's contaminant removal performance is given below.

In the course of the 5 years of operation, a number of operational issues and problems have become evident and required addressing. Principal amongst these is the anaerobic conditions which can occur from time to time, turning the pond completely black and foul smelling. It is unclear where the high biological oxygen demand (BOD) water suspected of causing this problem is sourced, although recycled beer cans are suspected. An aeration device has been installed at the inlet of the pond, to aerate the inflow, particularly under low flow conditions in summer. This has largely overcome the problem, although close monitoring of the condition of the pond is required, as a "crash" can occur relatively suddenly.

Water from immediately above the outflow is collected and used on site for dust suppression during dry periods.

Further enhancements to improve sediment removal due to the presence of elevated PCB concentrations, previously undetected and which have come to light due to toxicity screening of the effluent, have recently resulted in the installation of two continuous cartridge filters at the outlet of the pond. This is discussed in more detail below.

Stormwater Pond Monitoring

As indicated, the pond has been the subject of a range of monitoring since its construction. The results of the specific monitoring investigations are outlined below.

Contaminant Removal Efficiency

Intensive performance monitoring of the treatment pond was carried out initially, quantifying the mean concentration reduction and the event mean concentration for various contaminants. This early monitoring estimated similar removal efficiencies for each method (Leersnyder, 1993). Therefore, long-term pond efficiency monitoring has utilised mean concentration reductions on a monthly basis.

Table 2 compares the mean concentration reduction of the pond during the initial two year commissioning period (1992), the subsequent 1994-1996 period, and during recent toxicity studies (1997) undertaken on the pond. The 1992 and 1997 studies sampled pond performance over four individual storm events each.

It is worth noting the relatively high removal efficiencies maintained since commissioning except for soluble Zn. This is in spite of the progressive increase in contaminant loads to the pond since the initial commissioning period. The variability in pond performance is presumably related to the influent variability found in Table 1. However, the long term performance is considered to be acceptable from an engineering feasibility point of view. The toxicity of the inflow and outflows under these conditions and the impact on the wider environment are discussed below.

Table 2. Mean Concentration Reduction, %

Parameter	Initial Commisioning Period (1992)	Monthly Sampling (1994-1996)	Toxicity Testing (1997)
Suspended Solids	80	73	88
Copper (total)	97	79	92
Zinc (total)	92	98	95
Zinc (soluble)	85	78	31
Lead (total)	97	87	94
COD	9	31	ND
Oil and Grease	ND	9	ND

ND - Not Determined

Information relating to the relative contaminant removal efficiencies of both the settlement pond and the constructed wetland are illustrated in Table 3.

Wetland Sediment and Plant Tissue Quality

A range of monitoring investigations were conducted on the wetland component of the treatment device some 3 years after commissioning. These investigations included wetland conditions, the present concentrations of metals and total petroleum hydrocarbons in the wetland sediment, and the levels of metals in the dominant species of wetland plants (Bioresearches 1996). The results are discussed broadly below.

1. Wetland Condition

The wetland was commissioned with six species of plants including:

- Juncus articulatus
- Baumea articulata
- Cotula coronopifolia
- Eleocharis sphacelata
- Schoenoplectus validus
- Baumea juncea

At the time of sampling (3-1/2 years after planting), open water occupied approximately 50% of the total area of the wetland compartment. The wetland has subsequently developed further and now covers almost the total area.

Of the six original plants, it appears the taller plants were the most common, with the tall sedge *B. articulata* having the greatest cover. The smaller sedge *B. juncea* was infrequent, while *C. coronopifolia*, which was expected to occur around the taller plants, was totally absent.

In general, the wetland was in a healthy condition and the species diversity in the wetland had increased mark-

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Table 3. Percentage Contaminant Removal Efficiencies Showing the Relative Proportions Removed by Both the Settlement Pond and the Constructed Wetland Components (1994-1996)

	Suspended Solids	Copper	Zinc (Total)	Zinc (Soluble)	Lead	COD
Reduction Inlet to Outlet	73%	79%	98%	78%	87%	31%
Proportion Removed by Pond	93%	89%	82%	52%	87%	94%
Proportion Removed by Wetland	7%	11%	18%	48%	13%	6%

edly with the introduction of opportunistic plants, since commissioning. The wetland is also utilised by birds such as pukeko (swamp hen), shags (cormorants), and ducks. Maintenance was recommended, and subsequently undertaken to remove small colonies of the invasive grass species *Glyceria maxima* and raupo *Typha orientalis* for fear of their rapid spread and potential to exclude other rushes and sedges.

2. Sediment Grain Size

Generally the wetland substrate, which had accumulated since commissioning, was dominated by sediments less than .15 mm in diameter (i.e., very fine sands, silts, and clays). For sediments at the inlet end, the proportion of this finer material was 71.5% of the recently accumulated sediments, while the outlet end sediments contained 79.4% of the fine material. The greater difference was due to markedly higher clay concentrations at the outlet end of the wetland, reflecting the wetland's ability to remove some of the smaller sized particles. Based on these findings, it was calculated that the wetland was responsible for about 9% of the overall sediment removal. This agrees well with the estimates based on the long-term monthly sampling (Table 3).

3. Sediment Quality

The sediment quality information indicates that significant quantities of contaminants are being retained in the wetland, and that during the early life of the wetland, removal at the inlet end would appear to be more rapid. Further testing indicated that all the differences between wetland inlet and outlet sediment concentrations were statistically significant. The percentages of constituents at the outlet end in comparison with those at the inlet end were:

cadmium	23.5%
copper	37.5%
lead	21.2%
zinc	21.9%
total petroleum hydrocarbons	24.5%

With further analysis of the data it became clear that particular areas within the wetland contained higher levels of contaminants than others. The mounded areas introduced in the original design to "baffle" flows, appear to have increased channelling and short circuiting through the wetland. As a result of these findings, more attention is being given to the placement of such structures in future wetland designs.

Wetland sediment quality was compared to sediment quality guidelines for the protection of freshwater aquatic life (Persaud et al., 1992). The results indicate that 50% of the readings were above the lowest effects level (LEL) but below the severe effects level (SEL). While some 47.6% of results were above SEL levels, with the greater exceedance of SEL levels occurring at the inlet end (Bioresearches 1996). At present, there is no indication of abnormal plant growth or dieback which could be attributed to an increase in contaminant levels, and none would be expected given the generally higher level of robustness of these aquatic plants over other aquatic organisms and the limited uptake of contaminants by the plants themselves, as described below.

Although it is not possible to relate sediment contaminant levels to precise effects on the dominant wetland rushes and sedges, changes to the existing biological condition are expected to occur in a progressive fashion through time. A regular programme of monitoring (i.e. sediment levels, tissue levels, and plant vigour) is recommended to quantify this change and provide information on the optimum replacement intervals for the wetland.

4. Plant Tissue Quality

Plant tissue of the wetland plants were analysed for heavy metal concentrations. Table 4 summarises the average levels for all species. The data indicate that the highest concentrations occurred in *Juncus articulatus*, and that total metal loads are dominated by zinc concentrations, which are an order of magnitude higher than the other metal levels measured.

Unfortunately, no baseline data on tissue concentrations was collected prior to planting. However the plant tissue concentrations were compared to "control" plants from the broad geographical area of the original stock, and collected from areas outside of major industrial or other anthroprogenic influences. The assumption is that the "control" plants reflect broadly the metal concentrations of the plants prior to planting. Given these assumptions, the plant tissue concentrations outlined in Table 4 were found to reflect the pattern of concentrations in the "control" plants (Bioresearches 1996).

The highest average increase per species has occurred in the tissues of *Juncus articulatus* and *Baumea juncea* (13 times and 9 times, respectively) while the remainder of the plant species have shown an increase of 4 to 5 times.

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Table 4. Average Metal Levels in Stormwater Wetland Plants

Plant Species	Cadmium	Chromium	Copper	Lead	Mercury	Nickel	Zinc
J. articulatus	0.22	4.03	38.3	31.0	0.13	4.9	516.0
B. juncea	<1.033	1.53	2.7	3.6	0.29	1.1	125.7
S. validus	<0.029	0.73	4.6	0.71	0.024	1.1	91.0
E. sphacelata	<0.015	0.28	4.2	0.85	0.17	1.1	71.3
B. articulata	<0.058	0.43	2.3	1.32	0.016	1.0	39.3
Average	<0.071	1.4	10.4	7.5	0.13	1.8	168.7

With the inlet and outlet areas combined, the average plant metal concentration is only 8% of that found in the sediments; i.e., the accumulation of metals in the sediments is not paralleled by a similar rate of uptake by the plants, and the major mechanisms of metal removal in the wetland appear to be physico-chemical (flocculation, adsorption, settlement, filtration by plant stems) rather than biological.

Influent/Outflow Toxicity Testing

An extensive range of toxicity tests was conducted on both the influent and outflow from the treatment pond (NIWA 1997). The study measured influent and outflow chemical contaminant levels under storm conditions, and toxicity to a variety of freshwater and marine species. In addition, suspended particulate matter (SPM) in the outflow was collected by means of a centrifuge and used to assess the toxicity to sediment-dwelling marine species. Together, these chemical and toxicological components allow a risk assessment to be undertaken for potential ecological impacts on the marine receiving water environment.

1. Water Column Toxicity

The results of the pond water toxicity showed relatively high variability of the influent and outflow toxicity, although there was an observed decline in toxicity across the pond, with the Pacific Steel outflow generally classified as "slightly toxic" to "moderately toxic." In addition, the marine species appeared to be more sensitive than the freshwater, with the greatest response exhibited by a marine diatom and echinoderm.

Dilutions required to mitigate the outflow toxicity ranged from 11 to 46-fold (based on algae and echinoderms, respectively). These results suggest that of the test species currently available the algae and echinoderms, are the more sensitive species for monitoring stormwater impacts on the marine environment of Auckland.

Comparisons between the measured water column toxicity and the expected toxicity based on chemical analysis (totals) showed the measured toxicity to be much lower than expected. Acute (short term) toxic effects would have been expected for both the inlet and outflow on most occasions, based on the guideline exceedances for a number of metals. Similarly, some toxicity was observed on occasions when criteria were not exceeded, suggesting that contaminants other than metals may be contributing to the measured toxicity. Concentrations of ammonia contributed a maximum of 42% to the observed outflow toxicity, based on comparison with published chronic exposure guidelines.

Total and soluble contaminant concentrations are compared with acute and chronic freshwater and marine criteria in Table 5. The acute and chronic metal criteria exceedances over the four storm events monitored are shown in Figures 5 and 6, respectively.

The low measured water column toxicity suggests that the bioavailability of contaminants in both the total and soluble phases is low. One theory is that the high dissolved carbon values in the pond may have markedly reduced the bioavailability of the metals and subsequent measured toxicity on most occasions. Further work is programmed to look at this.

2. Sediment Toxicity

The objective of the sediment toxicity tests was to assess the toxicity of contaminants adsorbed onto the suspended particulate material (SPM), which passes through the pond, on benthic estuarine/marine organisms. Sediment samples were obtained from centrifuging the discharge of the pond.

The first test scenario attempted to simulate a "worst case" scenario appropriate to Auckland of 3 mm of annual contaminated estuarine sediment deposition. The second scenario involved the dilution of the SPM sediment with clean estuarine mud to simulate a likely deposition scenario, whereby discharged material is reworked with the surface layer of the estuarine sediments. SPM dilutions of 5x, 15x, and 50x were used.

The sediment toxicity under the "worst case" scenario showed low acute survival for amphipods and juvenile shellfish. However, given the high particulate carbon and nitrogen levels in the SPM sediments, it is unclear whether the observed toxicity was due to the contaminant levels or the anoxic conditions which developed in the sediments causing them to turn black and smell strongly of hydrogen sulphide. Microscopic examination of the SPM sediments showed high numbers of algal cells and protozoa present.

The acute survival for juvenile shellfish in the second series of tests showed no adverse effects at all dilutions.

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Table 5. Total and Soluble Concentrations (g/m³) of Metals in Water Samples used in Toxicity Tests with Acute and Chronic Exceedances

		Site	Fe Soluble	Fe Total	MN Soluble	MN Total	Cd Soluble	Cd Total	Cu Soluble	Cu Total	Pb Soluble	Pb Total	Zn Soluble	Zn Total
1	Storm Event 1	UI	0.168	0.189	0.0088	0.0583	0.00007	0.00014	0.058^{FM}	0.084^{FM}	0.0121^{FM}	0.1378^{FM}	0.153^{FM}	0.189^{FM}
2		UO	0.082	0.556	0.0004	0.0256	<0.00002	0.00012	0.016^{FM}	0.027^{FM}	<0.0005	0.0045^{F}	0.023	0.031
3	Storm Event 2	UI	0.09	1^{F}	0.0134	0.032	0.00017	0.0003	0.0098^{FM}	0.017^{FM}	0.0024	0.0257^{FM}	0.106^{FM}	0.144^{FM}
4		UO	0.47	1.9^{F}	0.023	0.173	0.00021	0.0004	0.0015	0.002	0.002	0.0063	0.21	0.027
5	Storm Event 3	UI	0.37	2^{F}	0.0996	0.175	<0.00005	0.0021^{F}	0.019^{FM}	0.036^{FM}	0.002^{F}	0.0418^{FM}	0.055^{FM}	0.483^{FM}
6		UO	0.13	1.8^{F}	0.829	0.51	<0.00005	<0.0001	0.0008	0.002	<0.0001	0.0014	0.017	0.04
7	Storm Event 4	UI	0.09	3.5^{F}	0.0068	0.117	<0.00005	0.0003	0.0082^{FM}	0.026^{FM}	0.0035^{F}	0.0643^{FM}	0.088	0.292^{FM}
8		UO	0.35	1.1^{F}	0.0089	0.098	<0.00005	<0.00001	0.0031^{FM}	0.004^{FM}	0.001^{F}	0.004^{F}	0.026	0.032
9	Storm Event1	PSI	0.149	36.7^{F}	0.0097	1.926	0.00042	0.028^{FM}	0.071^{FM}	0.961^{FM}	0.0172^{FM}	2.924^{FM}	0.131^{FM}	19.56^{FM}
10		PSO	0.31	1.97^{F}	0.0007	0.014	<0.00002	0.0005	0.017^{FM}	0.044^{FM}	<0.0005	0.0284^{FM}	0.053	0.238^{FM}
11	Storm Event 2	PSI	0.08	25.5^{F}	0.0061	0.981	0.00019	0.0043^{F}	0.0222^{FM}	0.428^{FM}	0.0063^{F}	0.812^{FM}	0.025	3.1^{F}
12		PSO	0.21	2.7^{F}	0.11	0.159	0.0005	0.001	0.0028^{M}	0.038^{FM}	0.0054^{F}	0.0777^{FM}	0.096^{FM}	0.372^{FM}
13	Storm Event 3	PSI	0.5	73.1^{F}	0.131	2.06	0.00159^{F}	0.0012^{F}	0.0127^{FM}	0.146^{FM}	0.0059^{FM}	0.32^{FM}	0.34^{FM}	1.75^{FM}
14		PSO	0.46	1.8^{F}	0.136	0.184	0.00078	0.0008	0.001	0.008^{M}	0.0041^{F}	0.0208	0.07	0.162^{FM}
15	Storm Event 4	PSI	<0.05	18.5^{F}	0.0111	0.795	0.00021	0.0065^{F}	0.0109^{FM}	0.382^{FM}	0.0025^{F}	0.974^{FM}	0.032	5.1^{FM}
16		PSO	0.11	3.7^{F}	0.123	0.233	0.00028	0.001	0.0056^{FM}	0.061^{FM}	0.0067^{F}	0.164^{FM}	0.145^{FM}	0.679^{FM}

[ ]	Exceedance of chronic criteria
F	Freshwater criteria
M	Marine criteria
FM	Exceedance of chronic criteria
FM*	Exceedance of chronic criteria
FM	Exceedance of acute freshwater criteria and chronic marine criteria

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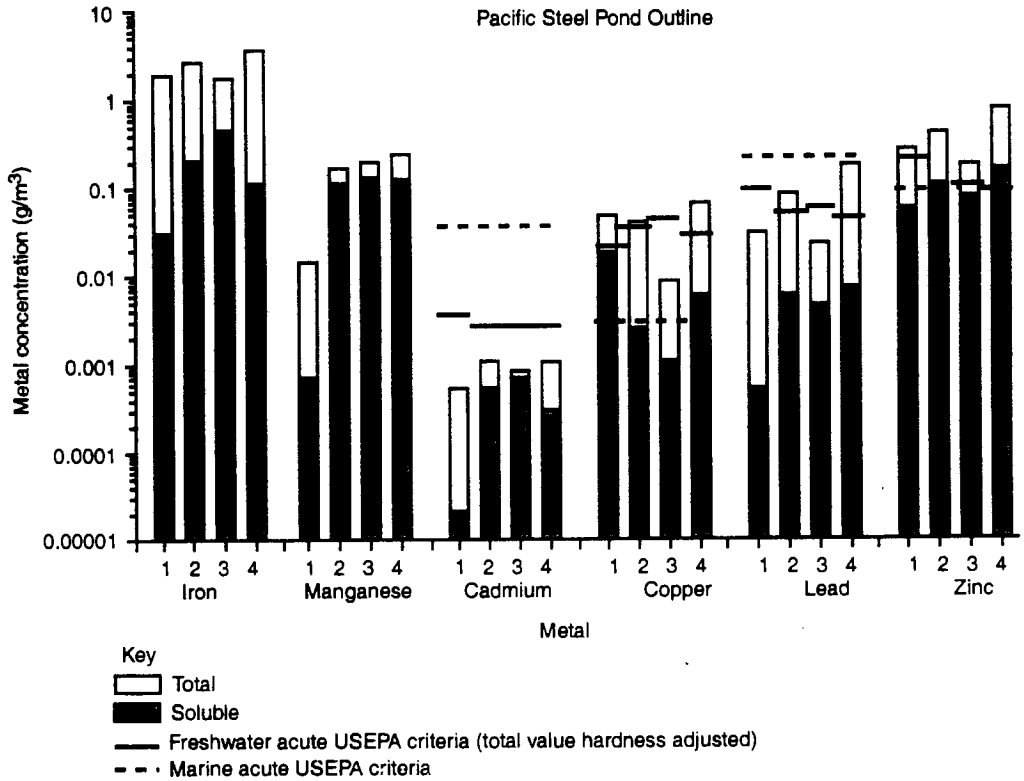

Figure 5. Metal concentrations with acute criteria exceedances in the Pacific Steel pond outlet over four storm events.

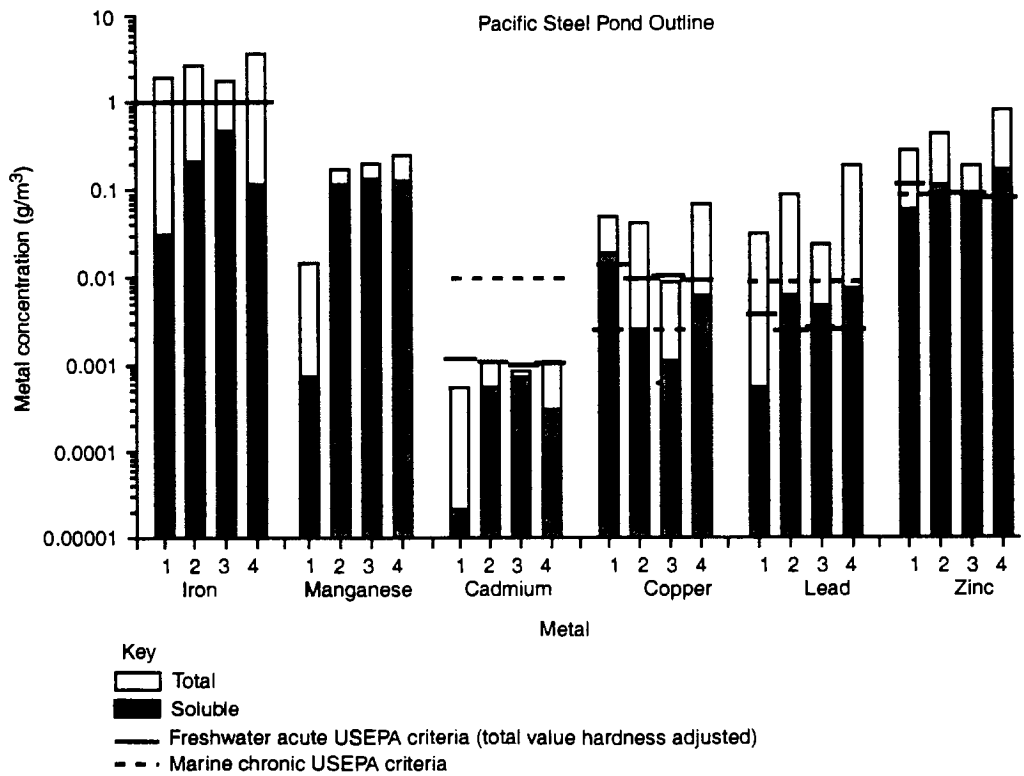

Figure 6. Metal concentration with chronic criteria exceedances in the Pacific Steel pond outlet over four storm events.

The amphipods, however, displayed threshold effects and enabled extrapolation of an EC50 response level of 70% SPM, and prediction of a threshold response (nominal EC10 response) concentration of 10% SPM.

Contaminant levels in the SPM sediments exceeded median effects sediment guideline levels (ER-Ms) for most organic and inorganic contaminants in the pond (Table 6).

Interestingly, Pacific Steel SPM sediments were also relatively enriched in some metals (Cd, Hg, Pb), chlordane, and polyaromatic hydrocarbons (PAHs) when compared with sediment samples taken from within the pond near the outlet. This suggests that there may be higher levels of some contaminants exported from the pond than would be indicated by comparative size fractions in the sediments near the outlet.

The relative toxicity of the various contaminants was compared by dividing the measured SPM concentration by the median effects guidelines (ER-M). The ratios indicate that for Pacific Steel, PCBs followed by DDTs represent the greatest organic contaminant risk, with zinc and lead the more potentially toxic metals. As a result of the elevated PCB concentrations measured during the toxicity investigations, considerable effort has been expended by Pacific Steel to identify the sources of PCBs on site and to prevent them from entering the stormwater system. In addition to the control of PCB contamination throughout the site, a pilot Mikroclean 1 micron nominal cartridge filtration device has recently been installed on the outlet to filter the discharge continuously. The performance of this filtration device is expected to result in a reduction of average suspended solids and PCB concentrations in the discharge of approximately 60%. These measures do not cover the discharge from the emergency overflow, which operates under larger, less frequent storm events.

The measured toxicity responses were in general agreement with the expected effects given the contaminant levels found in the sediments. Given this, the receiving water sediment dilution required to prevent significant adverse effects was estimated based on the chemical data and the guideline exceedence, together with the toxicity results. This suggests that a sediment dilution of up to 115x for sediments leaving the Pacific Steel Pond would be required to pevent significant adverse effects.

Biological Survey of the Receiving Environment

Ecological surveys of the tidal inlet which acts as the receiving environment for the stormwater treatment pond discharge, were undertaken in 1991 and 1995. Samples were collected for benthic invertebrate analysis and descriptions made of the marine vegetation present, as well as the use of the inlet by birds and fishes.

The southern inlet has a former Pacific Steel reclamation on the northern side while the southern side contains a recreational area, disused industrial land, and hospital land at the southwestern corner. Midway along the reclamation boundary is an inlet which receives inflow from a weired creek which drains a residential area. The inlet also receives stormwater from a heavy industrial area

Prior to the construction of the pond, the southern inlet acted as a natural stilling basin for both the Pacific Steel site and the stormwater discharges from the heavy industrial catchment above. The presence of saline conditions are thought to have increased the sediment deposition, and chemical reactions, probably with sea water, had resulted in the formation of precipitates which also settled out. (Bioresearches 1988). Sediment concentrations found in the inlet were elevated in comparison with the Mangere Inlet and the wider Manukau harbour. The inlet was characterised in 1991 as "heavily polluted" (Roan 1991)

In comparison with the conditions found in 1991, by 1995 the volume of freshwater entering the inlet had greatly increased. Contaminants had also declined due to Pacific Steel treatment pond and the closure of much of the heavy industry in the upper catchment. The total number of macroinvertebrate taxa recorded in 1995 was 25, compared to five in 1991. In 1991, the five species identified were typical estuarine organisms; however, the majority of those recorded in 1995 typically inhabit areas which are predominantly freshwater habitats (Bioresearches 1996a).

By 1995, much of the bare mud had been colonised by a variety of wetland/salt marsh plants, including *Cotula* and a variety of rushes. This recolonisation is reinstating a transitional zone more typical of inlets in other parts of the Mangere Inlet. It is also notable that this process is occurring despite the high levels of contamination in the underlying sediments (Bioresearches 1996a).

Following the 1995 survey it was concluded that *"there was no indication that the treated stormwater discharge has had a detrimental effect on the Inlet's biological condition, but has probably led to a more rapid development of a predominantly freshwater fauna upstream from the mangrove zone"* (Bioresearches 1996a).

Conclusion

The quality of the Pacific Steel stormwater discharge has improved dramatically since the commissioning of the stormwater treatment ponds in 1992. The performance of the ponds themselves has matched and even exceeded their predicted design efficiencies. The treatment system has settled down and is operating as expected, although day-to-day management of the entire stormwater system is required to ensure peak performance. At present, there appears to be no observable adverse effect on the wetland plants, and additional performance benefits have been clearly demonstrated from the inclusion of a constructed wetland in the overall stormwater treatment device.

The immediate receiving environment has also improved considerably in the past five years, as a result of both the improvement of stormwater discharges and land use

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Table 6. Comparison of Stormwater Sediment Contaminant Levels with Manukau Harbour Levels and Sediment Quality Guidelines (after NIWA 1997).

Stormwater comparison - total metals (mg/kgDW)

Site	C	N	Zinc	Cadmium	Copper	Mercury	Lead	Cumulative ER-M exceedance
Pacific - SPM	15.7	2.2	**7820**	16.7	**653**	**1.54**	**1430**	
Pacific - pond	5.1	0.5	**5635**	*8*	**1702**	0.14	**724**	
Reference Sediment Reglan	2.4	0.23	89.8	0.11	21.8	0.093	22	
Harbour site Comparison[c] Manukau - W	0.73 (26% mud)	-	86	-	19.4	-	6.1	
ER-L[d]			150	1.2	34	0.15	46.7	
ER-M			410	9.6	270	0.71	218	
Scenario I (undiluted) Pacific SPM/ER-M			19.0	1.7	2.4	2.2	6.6	32
Scenario II (diluted 5x) Pacific SPM/ER-M			3.8	0.34	0.48	0.44	1.3	6.4

Site	C	N	Tot PCB	DDTs	Chlordanes	PAHs	Dieldrin	Cumulative ER-M exceedance
Pacific - SPM[a]	15.7	2.2	**10753**	*44.2*	**55**	*5353*	**19.5**	
Pacific - pond[b]	5.1	0.5	**10500***	<50	<6	*1900*	<40.1	
Reference Sediment Reglan	2.4	0.23	0	0.1	0.1	112	0.1	
Harbour site comparison[c] Manukau - W	0.73 (26% mud)	-	2	22	*0.41*	*5311*	0.41	
ER-L[d]			22.7	1.6	0.5	1700	0.02	
ER-M			180	46	6	9600	8.1	
Scenario I (undiluted) Pacific SPM/ER-M			59.8	9.6	9.2	0.6	2.4	82
Scenario II (diluted 5x)			12	1.9	1.8	0.12	0.48	16

[a] SPM = suspended particulate matter, sediment collected by centrifuge
[b] pond sediment data from the 1995 survey (Nipper *et al.*, 1995)
[c] Data from Holland *et al.*, (1993)
[d] sediment guideline values from Long *et al.*, 1995, Long & Morgan 1991 (for chlordane and dieldrin)
* G. Mills, NIWA, personal communication
Key = **bold** = exceedance of ER-M values; *italic* = exceedance of ER-L values

changes which have occurred in the catchment above. The nature of the receiving environment has changed significantly to one dominated by freshwater inflows, primarily as a result of the Pacific Steel treatment pond discharge. The immediate receiving environment is not displaying any significant adverse effects as a result of the treated stormwater discharges, although should be remembered that the area had been classified as "heavily polluted" in 1992, and that the improvements made to date have to be compared with that impacted status.

With regard to the long-term implications of this treatment device, the results of the toxicity testing paint a somewhat more confusing picture. Measured water column toxicity of the outflow was found to be only slight to moderate, even though the measured contaminant concentrations, and in particular soluble zinc and copper, exceeded both freshwater and marine acute US EPA criteria. The reason for these findings is being investigated further but it is clear that something is reducing the bioavailability of these contaminants. It is also uncertain if this effect on the bioavailability of the contaminants is a long-term phenomena, or whether the contaminant loads discharged from the pond will become available in the future, and potentially give rise to adverse effects in the wider receiving environment.

The measured acute toxicity of the particulate material leaving the pond showed low survival rates for all test organisms in the experiments attempting to test the worst

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case scenario. However the anoxic conditions which developed during the experiment are likely to have influenced the result. The acute toxicity shown by amphipods with increasing pond SPM sediment concentration suggests that the discharge from the pond may have a longer term impact on the wider receiving environment as contaminant concentration levels build up.

Interestingly, the finding of elevated PCBs in the discharge has led to a range of measures being implemented on site which should see the sediment and associated contaminant levels being discharged decline by as much as 60% in the future. Work is on-going to assess the effectiveness of these measures in the longer term.

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Real World Modelling
A Case Study of the Silver Lake Watershed Project

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Abstract

The past decade has brought an increasing realization that our aquatic resources are better managed at the watershed level. This often requires the application of watershed modelling to provide managers with the information they need for decision making. Once, this required the hardware and services of consultants specialized in the field, along with the associated costs. However, advances in technology have brought computers powerful enough to run such models to the desktop of virtually anyone. Additionally, just as the private sector has gone through the "downsizing era," government has also been pressured to do more with less. These two factors have combined in such a way that staff members from local governments are now being asked to perform these studies. In a perfect world, data would be readily available for such studies, and the modelling would proceed just like the text book examples. Unfortunately, this is rarely the case. Modelers are often faced with data gaps and other problems which may not even come to light until well into the modeling process. This presentation will address these issues in the context of a case study of a watershed management project conducted in the Silver Lake watershed in central Delaware. It is hoped that a review of the problems encountered and ultimate solutions to those problems will be helpful to other "part-time modellers" finding themselves in similar situations.

Background

The Dover Silver Lake (DSL) watershed is centrally located in the state (Figure 1). The 168-acre lake is within the city limits of Dover, the state capitol. However, much of the 20,000-acre watershed is in Kent County, outside the city limits. There are four major sub-basins or "branches" which drain to the lake. The Maidstone Branch proved to be a significant sub-basin in terms of the modelling effort, as will be discussed later in this paper.

The lake and watershed are within the Coastal Plain Region and display many of the problems associated with water bodies in this region. Excess nutrients, particularly nitrogen and phosphorus, coupled with typical low summer base flow conditions have led to blue-green algal blooms from the time the lake was first formed as a mill pond in the late 1700's. Bathymetric, sediment and in-lake water quality surveys indicate that the lake is well into the eutrophication process, occasionally reaching hypereutrophic conditions in the summer months.

The lake's fish community is dominated by less desirable rough fish, such as gizzard shad and carp, which constantly stir the bottom sediments while feeding. This mixing of the water column is further exacerbated by the fact that the lake is a popular boating area and is the only inland lake in the state which still permits water skiing.

These factors combine to prevent the lake from being used to its fullest potential. Sport fisherman bemoan the fact that there are not more game fish to be caught in the lake. Even those that are caught have consumption restrictions due to toxins found in fish tissue samples. Although the city maintains a small swimming and beach area at a park along the lake, it is closed so many times during the summer due to health concerns that the city is considering building a public pool instead. Review of health department records indicates these closings have increased over recent years. Finally, landowners along the lake have to endure the smell and unsightly scum associated with major blue-green algae blooms which inevitably occur every summer.

Although the existing land use is largely a mix of agriculture and woodland, planners have predicted the watershed is slated for population growth of up to 52% and household increases of up to 73% by the year 2020. Since much of this growth will occur outside the water and sewer service area provided by the City of Dover, it is expected that on-site waste treatment facilities in the porous, relatively high water table soils will be a major source of nitrogen loadings to the lake. Sedimentation from active construction sites, fertilizer applications to lawns, and pet wastes are also expected to be significant potential sources of nonpoint source (NPS) pollution as urbanization of the watershed increases. In the fringe areas where development has occurred, bioassessments have already indicated evi-

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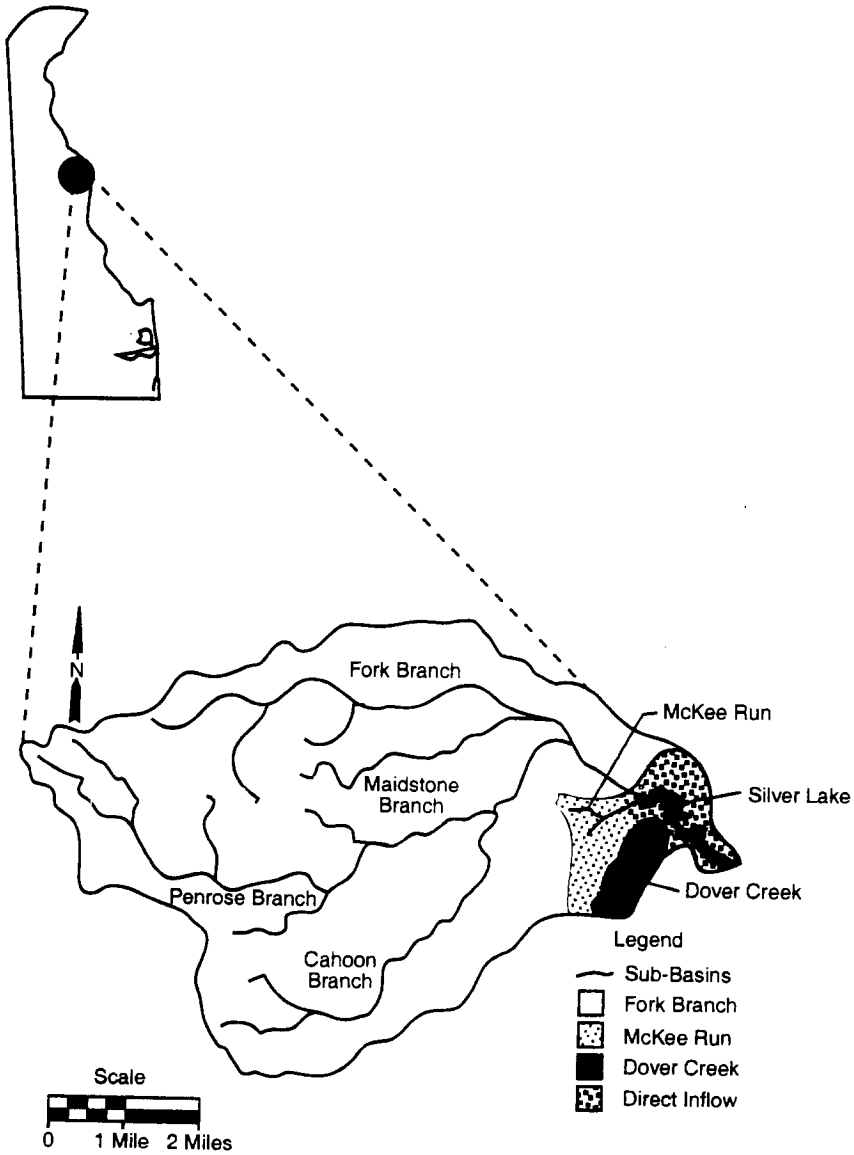

Figure 1. Dover Silver Lake Watershed

dence of impacts to stream systems due to higher stormwater runoff volume and longer duration of flow. Based on these projections, impacts to the lake and streams in the watershed are not likely to subside and may increase.

Initiation of the Modelling Effort

The City of Dover first approached the Department of Natural Resources and Environmental Control (Department) with documented water quality problems in the lake. However, it was recognized that Kent County would have to be a major player involving any recommendations dealing with land use. An advisory committee was formed, with representatives from the department, city, county, environmental groups, builders/developers and other stakeholders. It was decided that a watershed protection/restoration plan was needed to keep the lake from further degradation. An important component would be the development of a watershed model, which would allow decision-makers to assess impacts to the lake based on various policies and recommendations put forward in the plan.

Although a grant was procured from the National Oceanic and Atmospheric Administration (NOAA), it was apparent that there would not be enough funding to contract the modelling to an outside consultant. At this point, the advisory committee enlisted the help of the department to assist in the modelling effort. Department staff felt it would be feasible to develop a "planning level" model, but that a "deterministic" model would require additional resources. It was decided that a "planning level" model would suffice,

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and the staff were told to proceed. The remainder of this paper will describe some of the goals and methodologies of the DSL modelling effort. Although these have been separated into data issues and model issues for purposes of discussion, the reader should be aware that many of these issues must be addressed concurrently during the modelling effort.

Data Issues

Collection

In most cases, data collection will be the first priority in model development. A thorough review of existing information and data should be done. Sources of such information used in the DSL Watershed Project included:

- Previous studies done in the watershed under the Clean Lakes Program
- An analysis of the lake dam conducted by the Corps of Engineers (USACOE)
- A flood study conducted by the Federal Emergency Management Agency (FEMA)
- Water quality data collected in the watershed as part of the department's responsibility under the Environmental Protection Agency's (USEPA) 305b report
- Soils information from the Natural Resources Conservation Service's (NRCS) soil survey for Kent County, DE
- Land use data from the department's GIS database
- Zoning maps from the City of Dover and Kent County
- Population projections from the State of Delaware
- Topography based on US Geological Survey (USGS) 7.5-minute quadrangles
- Rainfall data from the USGS
- Flow data from the USGS

The last two items are especially important in any watershed modelling effort. As will be discussed, it is important to realize that in most cases, it will be necessary to have two years' worth of rainfall and flow data for the calibration/verification process. The data must also be available in a format which is usable in the chosen model. For example, use of a continuous-type hydrologic model will require rainfall and flow data in a time series format, such as 15-minute interval recordings. Daily totals would not be adequate for use with such models.

Gaps

Data gaps are almost certain to occur in any major watershed modelling effort. Filling those gaps is, of course, dependent on the nature of the missing data. Gaps in rainfall and/or flow data must be addressed quickly, since modelling cannot proceed without such data. As mentioned previously, it is desirable to have two years' worth of data for the hydrologic model. Obviously, if data are not already available and must be collected, the modelling itself must be put on hold until the data become available. Therefore, rainfall and flow data can be considered "mission critical" and should be among the first data collected.

For the DSL Watershed Project, rain gages were placed in two locations within the watershed at project inception to check for spatial and temporal variation in rainfall. In addition, the USGS maintains a time-series rain gage at its main office in Dover. A statistical analysis was conducted on the data from the three gages to ensure that there were no significant differences in the rainfall data. Once this was verified, missing data from one station could be compensated with data from another station with confidence.

Although it was desired to have stream gages located at all four of the major branches feeding the lake, funding only allowed contracting with the USGS to re-establish a gage on the Maidstone Branch; however, the USGS maintains a gage below the Silver Lake dam which measures the flow from the entire watershed and has been in continuous service for more than 50 years.

Although land use data were available for the watershed, they were not a perfect match with some of the land uses used to establish pollutant loadings. Specifically, the initial land use map lumped all agricultural land together. Since pollutant loadings vary considerably among agricultural land uses, it was felt that further breakdown of this category would be necessary. Fortunately, the DSL watershed is immediately adjacent to a sub-watershed which drains to the Chesapeake Bay. The Chesapeake Bay Program (CBP) has done extensive data collection for modelling purposes. Land use data were collected from a remote satellite platform as part of the USEPA EMAP Program. Further, the CBP data had gone through a field truthing QA/QC process. Field investigation indicated that the agricultural land use in the CBP's Choptank Segment was similar to that of the DSL watershed. This made it possible to use data from the Choptank Segment for extrapolation purposes in defining the various agricultural land uses in the DSL watershed.

Ambient water quality data were available from several STORET stations located in the watershed; however, there was very little water quality data collected from runoff during actual storm events. Initially the project team intended to implement a stormwater runoff sampling program to provide this data. However, this proved to be a very costly venture in terms of equipment, lab costs, and staff time. Furthermore, the year the data were to be collected turned out to be one of the driest in many years. It was decided that an alternative was needed. One of the areas studied during the National Urban Runoff Program (NURP) was the Metropolitan Washington, DC area. This study became the basis for the DSL urban land use water quality data. Once again, the Chesapeake Bay Program provided wa-

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ter quality data from agricultural land uses. Locally collected stormwater data were used for verification purposes.

Analysis

One of the most crucial aspects of the modelling process is data analysis. This is the opportunity for the modeller to get to know and understand the hydrologic processes of the watershed. This starts with an analysis of the rainfall. The modeller should compare the data being used for the model with historic data to make sure that there are no anomalies. Data used for the calibration run should also be compared to that used for the verification run. Variation in this data is to be expected. However, if the variation is large, it should be kept in mind that it may be difficult to verify the model with a high degree of confidence.

Arguably, the flow data are the most important and should be analyzed accordingly. The US Geologic Survey does an excellent job of summarizing data from its own gages in its annual "Water Resources Data" reports for both surface and groundwater. This provides a good "first cut" analysis but should be supplemented whenever possible by closer scrutiny of the data itself. In the case of the DSL Project, USGS time-series flow data were imported into a spreadsheet program and graphed so that individual storms could be analyzed. As mentioned previously, there were two gages in the watershed. One was below the spillway of the Silver Lake dam on the St. Jones River, while the other was on the Maidstone Branch, a major tributary draining to the lake.

Although is was desirable to model the watershed as a whole, the available flow data would make this difficult. The St. Jones gage measured the outflow from the lake and therefore represented flow which had been routed through a reservoir. Inflow data to the lake was limited to only one tributary, the Maidstone Branch. Furthermore, the St. Jones gage captured approximately 150 acres of highly urbanized land below the dam spillway which resulted in an incidental peak in the hydrograph prior to the main peak from the reservoir. This incidental peak, along with the reservoir effect made the St. Jones data less desirable from a watershed modelling standpoint. However, visual observation indicated that the flow data from the Maidstone Branch, though lower in magnitude, was clearly correlated to that for the St. Jones. Statistical analysis confirmed this correlation; therefore, the Maidstone Branch was used as a "surrogate" for the DSL watershed as a whole. This eliminated the problems associated with the St. Jones flow data and simplified the modelling process. The relationship with the highest correlation was the "Rv" value or runoff coefficient. This correlation was valuable in modelling the future land uses, as discussed later in this paper.

Analysis of the flow data also revealed some other relevant hydrologic relationships. There were considerable seasonal differences in the flow for similar storm events. For example, a storm on April 14, 1993 having 1.26" of rainfall created a peak discharge ten (10) times greater than a similar storm on July 14, 1993 having 1.50" of rainfall. This was apparently due in large part to the effect that vegetation had on interception and transpiration. It was also no doubt related to greater infiltration losses and available surface storage. Whatever the reason, this has important implications for modelling. If one were to rely only on event-type models to represent the hydrology of such a watershed, the results may differ significantly from reality.

Another important hydrologic relationship is the ratio of the surface runoff to the base flow. A manual separation of one year's flow data from the Maidstone Branch revealed that the base flow actually accounted for approximately 50% of the total flow. This also has important implications for the modelling. From a quantity standpoint, the model must have the capability to differentiate the surface flow and base flow in order for a true calibration to be accomplished. From a quality standpoint, the capability to account for the base flow is essential to estimate loadings from soluble constituents such as nitrogen.

Uncertainty

One of the most difficult tasks to deal with in any modelling effort that attempts to predict future impacts is that of uncertainty. For the DSL Project, there were two areas of uncertainty which had to be addressed: future land use and the hydrologic response due to this change in land use. While zoning maps often provide some basis for determining future land use, the unincorporated areas of the watershed were zoned almost entirely as agricultural-residential with an average density of 1 unit/acre on the existing maps. To complicate matters, the City of Dover and Kent County were both in the midst of updating their comprehensive plans, which would not be finalized until after the watershed project was scheduled to be completed. Although planners had predictions for population growth through the year 2020, the actual land use mix associated with that growth was anybody's guess. After wrestling with this dilemma for some time, staff decided to develop five alternative build-out scenarios of increasing density. The mix for these build-out scenarios was based on land use mixes observed in adjacent, more highly urbanized areas. Alternative 3 was considered the most likely build-out scenario for the watershed and consisted of the following land use mix:

- 40% undeveloped
- 25% low-density residential
- 20% medium-density residential
- 10% high-density residential
- 5% commercial/industrial/institutional

The various build-out scenarios, of course, provided the basis for modelling future impacts to the lake, both in terms of water quantity and water quality. Predicting changes in hydrology is one of the strengths of event-type modeling. However, for continuous-type modelling, some input parameters are used for calibration purposes that are not

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strictly based on physical characteristics of the watershed. Calibrating to an unknown condition is tenuous at best unless one has considerable experience with that model. Fortunately, one of the flow relationships was of value in overcoming this obstacle. As mentioned previously, the strongest correlation found during analysis of the flow data was the runoff coefficient or "Rv" value. This is merely the ratio of the runoff (r) to the precipitation (p) and can be expressed mathematically as follows:

$$Rv = r/p \quad (1)$$

This relationship was also examined during the NURP study using runoff data collected from the various study areas. Researchers further found that the runoff coefficient was related to the percent imperviousness (I) in the watershed. A regression analysis resulted in the following equation (adjusted $R^2 = 0.71$):

$$Rv = 0.05 + 0.009(I) \quad (2)$$

Since this relationship ties the runoff with the percent imperviousness, the DSL Project staff were able to use it as a basis for calibrating the continuous hydrologic model for the future build-out scenarios.

Variability

As anyone who has worked with water quality data is aware, there is a tremendous degree of variability in such data. While it is generally accepted that most constituents found in stormwater runoff exhibit a lognormal distribution, it still takes a relatively large data set to characterize local conditions within a comfortable level of confidence. This proved to be cost-prohibitive in the case of the DSL Project and was a major consideration in the staff's decision to develop a "planning level" model as opposed to a deterministic model. Statistical methodologies and Monte Carlo simulations are an alternative, but still require a high degree of confidence in the distribution function. In the end, a simple approach using low, median, and high values was used for the water quality constituents. This approach was also used for analyzing the effects of Best Management Practices (BMPs) in the watershed based on low, median and high removal efficiencies. A matrix was then developed for the various build-out scenarios, constituent loadings and BMP removal rates. From this, it was possible to assess impacts to the lake based on worst case, best case and most-probable case scenarios.

Model Issues

Decision

The decision as to whether to do watershed modelling should not be taken lightly. Modelling is a labor-intensive undertaking, requiring a considerable commitment of staff resources. In most cases, the time required for such an effort will be underestimated. Alternative methods should be explored whenever possible. Results from other watershed studies in the area may be just as valid for planning purposes. This could preclude having to do modelling altogether. In any case, modelling should be viewed as a "least preferred option" for assessing a watershed.

Selection

If the decision has been made to proceed with a modelling effort, the next step is to determine which model to use. This is usually based on what aspect the watershed study is focused on. However, in some cases the choice of a particular model may be predicated on the available data. If time series flow data are not available, for example, some models may not be an option. The capabilities of the modellers must also be considered. Models range in complexity from simple regression formulae to statistical models to computer programs capable of doing very sophisticated hydrologic simulations. There is, unfortunately, no single model currently developed which can be thought of as providing "one-stop shopping." Some models, for example, are geared to event-type modelling for assessing flooding and water quantity impacts. Others operate in continuous mode for assessing water quality impacts. Some model runoff conditions, others model receiving water interactions. The models used for the DSL Project were as follows:

- Hydrologic Models

  TR-20 (NRCS) - Single-event mode only, synthetic rainfall; used to estimate runoff associated with major storms (i.e., 10-YR, 100-YR, etc.) for FEMA floodplain studies.

  PCSWMM (EPA) - Single or continuous mode, actual rainfall; calibrated against actual flow records to simulate hydrologic processes for Maidstone Branch sub-watershed.

- Hydraulic Model

  HEC-RAS (COE) - Used to estimate water surface profiles for FEMA floodplain studies.

- Water Quality Models

  PCSWMM (EPA) - Used to estimate pollutant loadings for single or continuous rainfall events based on Event Mean Concentrations (EMC).

  Excel (Microsoft) - Spreadsheet program used to estimate pollutant loadings based on both EMCs and Export Coefficients.

- Receiving Water Model

  Vollenweider Model - Regression formula used to estimate the steady state phosphorus concentration in Silver Lake.

- Trophic State Model

  Carlson Trophic State Index (TSI) - Method used to determine the likelihood that the lake displays a particular trophic state (i.e., mesotrophic, eutrophic, etc.).

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Calibration/Verification

For hydrologic simulation computer programs such as EPA's Stormwater Management Model (SWMM), it is essential to calibrate the model to local data. If available, a full year's worth of rainfall and flow data in a time-series format would be preferred. The purpose of the calibration run is to make sure that the output from the simulation is consistent with observed data. This is largely a matter of adjusting the various input parameters until the results from the simulation agree with the observed data within an acceptable degree of variance. The modeller must be careful not to adjust the parameters outside the range of accepted values just to get a better calibration. The adage "garbage in, garbage out" certainly applies in this case.

Once the model is calibrated, a verification run should be made with an independent data set. The verification process assures that the model has not been merely optimized to a single data set, but can give consistently realistic results as the input data varies. The calibration and verification process has often been referred to as the "what is" stage of the model.

Prediction

Once the model has been calibrated and verified, it can be used for assessing "what if" scenarios. This is the primary purpose of any major watershed modelling effort. In the case of the DSL Project, future impacts to the tributaries and lake under various land use changes could be assessed, along with potential mitigation methods. Some of the results were as follows:

- Peak discharges, volume of flow, duration of flow, and out-of-bank incidences could all be expected to increase under the various build-out scenarios. The percentage increase depended on the percentage of imperviousness.
- Under the most probable build-out scenario, flow depth increased 9 inches for a storm which under existing conditions was just at bank-full elevation.
- As a result of the hydrologic changes, stream channel erosion would be expected to increase, thus increasing total suspended solids (TSS) loadings to the lake.
- Loadings of total phosphorus (TP) to the lake remained relatively flat with increased build-out.
- Loadings of total nitrogen (TN) to the lake increased slightly with increased build-out.
- Even under a best-case scenario, it was unlikely that the lake would drop below eutrophic conditions.

Validation

A step which is often overlooked in some watershed studies is validation of the model results. This is not to be confused with verification, which is more closely associated with the functioning of the model itself. The validation process is an attempt to assure that the results from the model can be accepted. A model could conceivably be calibrated and verified, yet still yield unacceptable results. The water quality estimates from the models developed for the DSL Project were validated by comparing them to observed concentrations from several STORET stations in the lake. The model results indicated that the average total phosphorus (TP) concentration in the lake for the existing condition would be 0.22 mg/l. This compared favorably to the average observed TP concentration of 0.17 mg/l, and was well within the range of observed values. Thus, staff felt the results from the models could be accepted, particularly for planning purposes.

Conclusions

As environmental managers become more aware of the need to manage water and aquatic resources at the watershed level, supporting staff are increasingly being asked to provide information for decision makers. This often includes the use of watershed modelling. Such was the case for the Dover Silver Lake Watershed Project located in central Delaware (DDNREC, 1992, 1995a, 1995b). Staff were enlisted to model a 20,000-acre watershed for the purpose of preparing a watershed protection/restoration plan. A critique was done at the completion of the project and it is hoped that information gleaned from that critique will be helpful to others finding themselves in similar situations. Basically, it was found that the modelling tasks tended to be grouped as either data issues or model issues. Data issues included collection, gaps, analysis, uncertainty and variability. Model issues were decision, selection, calibration/verification, prediction and validation.

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Water Quality Modeling to Support the Rouge River Restoration

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Abstract

The Rouge River National Wet Weather Demonstration Program (Rouge Project) has taken on the challenge of implementing river restoration efforts in a highly urbanized watershed. The 467-square mile Rouge River Watershed is located in southeastern Michigan, and encompasses 48 communities, including the City of Detroit. A significant number of stormwater and combined sewer overflow (CSO) controls are being installed within the watershed to address Rouge River pollution reduction objectives.

A suite of hydrologic, sewer system and riverine water quality models have been used to address technical questions that have been asked in Rouge River Watershed planning. This paper presents application of four of the models used by the Rouge Project: 1) *TRTSTORM*, 2) *Watershed Management Model (WMM)*, 3) *Stormwater Management Model (SWMM)*, and 4) *Water Quality Analysis Simulation Program (WASP)*. The TRTSTORM model predicts annual overflow statistics for various CSO control facilities. A simple pollutant loadings model, the WMM evaluates and communicates the relative impacts of various stormwater controls. SWMM is aiding the development of subwatershed management plans by predicting relative changes in wet weather river response for alternative controls. Finally, the WASP event model predicts the highly transient dissolved oxygen drops caused by CSO discharges, thus the benefits for various levels of CSO control.

The Rouge Project models have been and continue to be an important decision-making aid for the project. In addition, the modeling approach used by the Rouge Project, as well as several specific modeling tools, are transferrable to other urban watershed management projects.

Introduction

The Rouge Project is using four modeling tools to support river restoration efforts in the highly urbanized Rouge River Watershed. The U.S. Environmental Protection Agency (USEPA) sponsored the Rouge Project, in 1992, to demonstrate effective solutions to wet weather water quality problems in urban areas. Under the leadership of the Wayne County Department of Environment, CSO controls and stormwater best management practices (BMPs) are being implemented within a watershed approach which stresses an inclusive process of all stakeholders.

This paper presents an overview of how water quality models are being used to answer technical questions which arise in the Rouge Watershed planning process. Application of four specific models is discussed including each model's role and sample results which illustrate how the model could be applied in other watersheds. Several lessons learned in the Rouge Project modeling effort are also presented.

The Rouge Watershed

The Rouge Watershed encompasses 467 square miles in Michigan's greater Detroit metropolitan area and is home to 1.5 million residents. The Rouge River has been identified as one of the most polluted rivers in the Great Lakes basin. The Lower, Middle, Upper and Main Rouge River branches total 127 miles in length, and comprise one of the state's most publicly accessible rivers.

Multiple pollution sources have led to the gradual degradation of water quality and habitat in portions of the Rouge River and resulted in use impairments. The primary problems include CSOs, nonpoint stormwater runoff, illicit connections, failing septic tanks, stream bank erosion and increased flow variability. The combined effect of these pol-

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lutants has led to depressed DO levels, whole-body contact prohibitions, damaged aquatic habitat, fish consumption advisories and poor aesthetics.

One-third of the CSOs in the watershed are being controlled via 11 demonstration CSO basins, several of which became operational in 1997. Performance of the demonstration facilities will be used to determine the appropriate level of control for the remaining CSOs. The watershed has been divided into 11 subwatersheds where advisory groups are forming to address all other pollution sources in a holistic fashion. Numerous stormwater BMPs, recreation and habitat projects have already begun and more are planned.

Modeling Approach

The modeling effort consists of a three-tiered approach. Tier 1 consists of several small area models used to simulate flows, pollutant loads and concentrations from specific pilot projects or localized areas of study such as wetlands, swales, wet detention ponds and individual CSO basins. Tier 2 consists of a simple pollutant loading model and a detailed sewer system model that both simulate pollutant generation by subarea for the entire watershed. Tier 3 is the river models which simulate instream flows and concentrations in the four main river branches, based on the inputs from the Tier 2 detailed sewer system models. Following are four examples of these models in use.

CSO Facility Performance

While the 11 demonstration CSO facilities were in the design stages, the TRTSTORM model was developed to provide some early predictions as to how these basins would perform (Kluitenberg et al., 1994). The model was used to address the following questions:

- How will the proposed CSO facilities, designed to several different sizing criteria, perform relative to presumptive criteria in the USEPA CSO Policy (USEPA, 1994)?
- What annual pollutant load reductions are expected from the proposed facilities?

The TRTSTORM model is a simple hydrologic mass balance model which tracks CSO facility filling, treatment, overflow, dewatering and decanting based on long-term hourly precipitation records. It is a modified version of the U.S. Army Corps of Engineers "Storage, Treatment, Overflow, Runoff Model" (Hydrologic Engineering Center, 1976). The model generates annual performance statistics for flows to the treatment plant (via interceptor), treated and untreated overflows.

The model was used to show that all CSO facilities designed to the demonstration sizing criteria should meet the 85% capture and four overflow per year presumptive criteria in the USEPA CSO policy. The model used assumed treatment efficiencies to determine expected annual load reductions for a number of pollutants at each facility. Figure 1 shows a summary of the predicted annual reduction in biochemical oxygen demand (BOD) entering the receiving water for: one site-limited facility; five basins sized to provide 20-minute detention of a 1-year, 1-hour storm (demonstration size); and two basins sized to capture a 1-year, 1-hour storm (Michigan Department of Environmental Quality (MDEQ) size). The results make it clear that for either of the two sizing criterion, annual load reduction is strongly governed by capture and is fairly insensitive to basin treatment efficiencies.

Pollutant Loading Analysis

The Watershed Management Model (WMM) is being used to estimate annual pollutant loading. In each subwatershed, WMM is being used to address the following questions and to communicate technical findings to stakeholders in an easy-to-understand fashion.

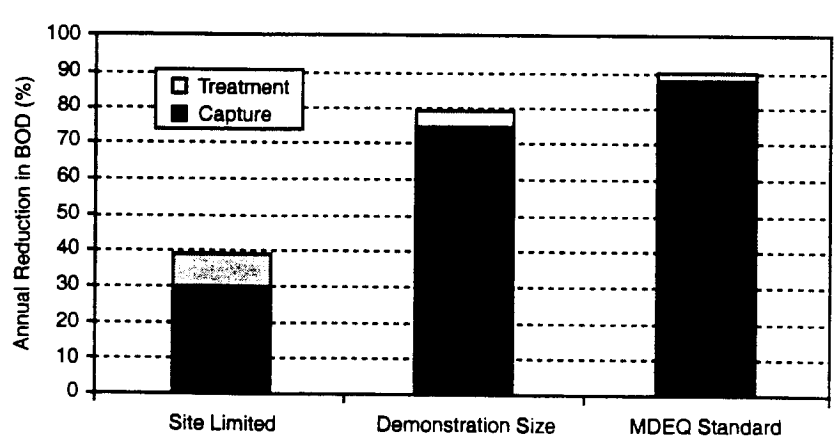

Figure 1. Annual percent reduction in BOD for various basin sizing criteria.

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- What are the relative contributions of different pollutant sources in the subwatershed?
- What pollutant load reductions can be expected with various stormwater BMPs and CSO controls?
- How will expected land use changes impact pollutant loads at the bottom of the subwatershed?

The Rouge Project recently completed development of WMM for Windows (Rouge Program Office, 1997) which is being provided to each community for its own use in subwatershed planning efforts. WMM calculates pollutants loads for each source of flow (baseflow, stormwater runoff, CSOs and point sources) in each watershed subarea using annual flow volumes and event mean concentrations (EMCs) assigned to that specific source. The model projects annual pollutant loads by subarea. Various combinations of stormwater BMPs and CSO controls can be selected in specific geographic areas to determine the overall resulting pollutant reductions for a particular management plan.

WMM was used early in the project as a prioritization tool to develop pie charts showing the major pollutant sources in each subwatershed. It was recently used as an analysis and communication tool in three detailed subwatershed management studies. Figure 2 is a sample of WMM results for BOD in the Middle 3 subwatershed, where it was used to show the cumulative effect of two-phase CSO control and two different stormwater management plans.

Watershed Hydrology/River Hydraulics

The Rouge Project has developed a continuous, growing-season model of the entire watershed and the major river branches using the USEPA Stormwater Management Model (SWMM) (Huber et al., 1992). The model is used as the hydraulic driver for the riverine water quality model. It has also been used to assess river hydraulic impacts for issues which arise in the subwatershed planning efforts. Questions it has addressed include:

- How will expected land use changes impact instream hydraulics (flow rates, volumes, depths and velocities)?
- How will proposed CSO control facilities impact instream hydraulics?
- What combination of stormwater BMPs and CSO controls will reduce instream peak flow rates to workable levels for suitable fish habitat?

The SWMM RUNOFF block is used to model the hydrology of all storm sewered areas and areas with natural drainage. An existing SWMM RUNOFF/TRANSPORT model, the Greater Detroit Regional Sewer System Model (Camp Dresser & McKee, Inc., June, 1994), is used to model all CSOs entering the river. Inflow hydrographs from both these models comprise all inputs to the one-dimensional river model, which is simulated with the SWMM TRANSPORT block. A continuous simulation with the full model was calibrated to 6 months of 15-minute data collected with a network of rain and stream flow gages. We assume that these data coincide with a 40- or 50-year analysis.

As part of the Upper 2 Subwatershed Management Study, the model was used to evaluate several scenarios including the cumulative impact of future land use projections, complete CSO control, placement of regional extended dry detention ponds throughout the subwatershed. A fourth scenario involved placement of such ponds at only a few select locations in the subwatershed instead of everywhere. The average increase in peak flow rates for a range of typical storms is shown in Figure 3 for one sample location. The results clearly show that the existing high

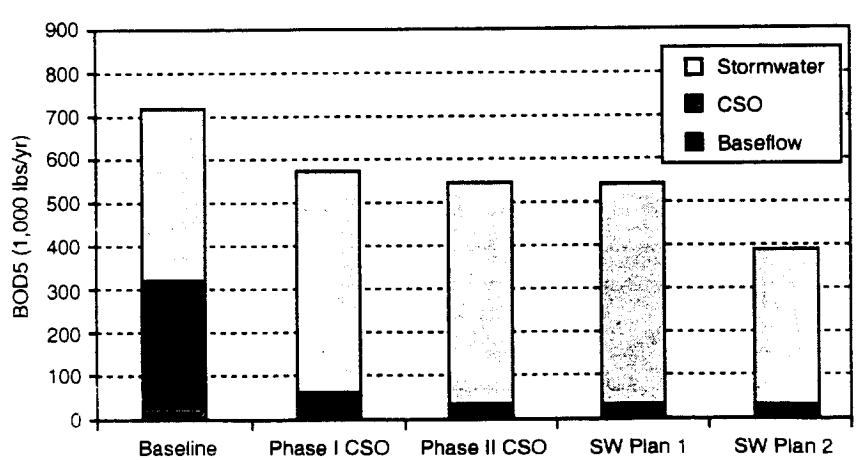

Figure 2. Middle 3 subwatershed WMM model results - average annual BOD load.

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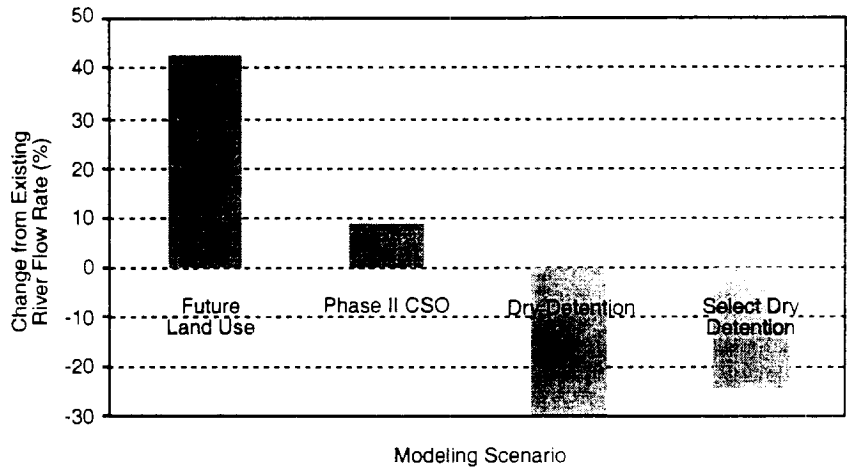

Figure 3. Model river flow rate compared to existing conditions - Upper 2 subwatershed - Bell Branch at Beech Daly.

flow rates and velocities and the resultant bank erosion problems will worsen, but that both types of regional detention could be used to accommodate future land use changes and reduce peak stream flows and velocities below existing conditions.

Instream Water Quality

Building on the SWMM quantity model, a riverine water quality model of the Lower, Middle, Upper and Main Rouge River branches was developed using the Water Quality Analysis Simulation Program (WASP) EUTRO model (Ambrose et al., 1993). While the model was originally developed and calibrated as a continuous model of eight pollutants, it has evolved to its current, primary role as an event model to simulate the CBOD-DO interaction which results from CSOs, including the sudden transitory DO drops which have been observed in the Rouge River. The model is currently being used to address the following questions:

- Will various CSO control alternatives eliminate the transitory DO drops caused by high CBOD in CSO discharges?
- What wet weather DO impairment will remain after all CSO controls are in place?
- How much will dry weather DO improve after controls eliminate most of the sediment oxygen demand (SOD) contributed by CSOs?

The water quality model developed is shown schematically in Figure 4. Stormwater inputs are simulated with the SWMM RUNOFF build-up/washoff algorithms. CSO inputs are assigned concentrations based on the time from when overflow begins, based on typical "pollutograph" shapes from monitoring data. The Rouge Project also developed a new model code linking the SWMM TRANSPORT river hydraulic output to the WASP model. Portions of the model have been calibrated to several heavily monitored wet weather events.

The model was utilized to evaluate two alternative CSO basin sizes in Oakland County on the main branch of the Rouge. For one of the calibrated wet weather events, the instream DO improvement was determined by modeling the impact of complete CSO control with three CSO basins sized to the demonstration criterion. The simulation was also repeated assuming the basins were enlarged to the MDEQ standard sizing criterion. The simulated instream DO shown in Figure 5 illustrates that the demonstration size basins improve the DO sag enough that it no longer falls below the 5 mg/l standard for this event. It also shows the marginal improvement which would have been achieved if the MDEQ basin sizing criterion were used, approximately doubling the size of each of the facilities.

The model of the entire main branch of the Rouge was also used to simulate dry weather DO, which is primarily driven by SOD and reaeration. For the first phase of CSO control and also for complete control, model SOD was reduced to approach that of in-situ SOD measurements made in river reaches which were not CSO impacted. The results in Figure 6 show that CSO controls will provide a significant benefit to dry weather DO, but that some DO impairment will remain in selected river reaches which are somewhat impounded.

Instream performance monitoring began in 1997. The monitoring is intended to show whether effluent from the demonstration facilities will cause any remaining water quality impairment. The water quality event models will be used in the future as part of the analysis of the monitoring results.

Model Findings

Many findings have arisen out of the Rouge Project, several of which the models helped bring to light. Model findings are given below.

- The impairments caused by wet weather pollution are certainly not limited to wet weather periods. In the Rouge this is especially true for the CSO contributions to SOD and the resultant dry weather DO impairment.

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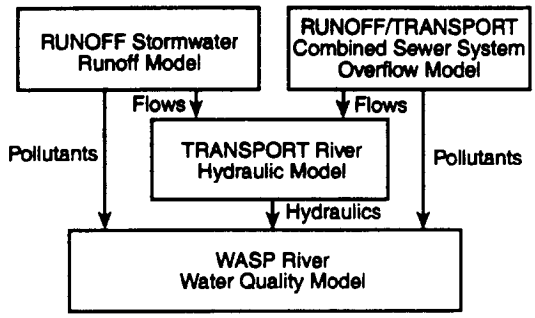

Figure 4. Rouge Tier 3 model schematic.

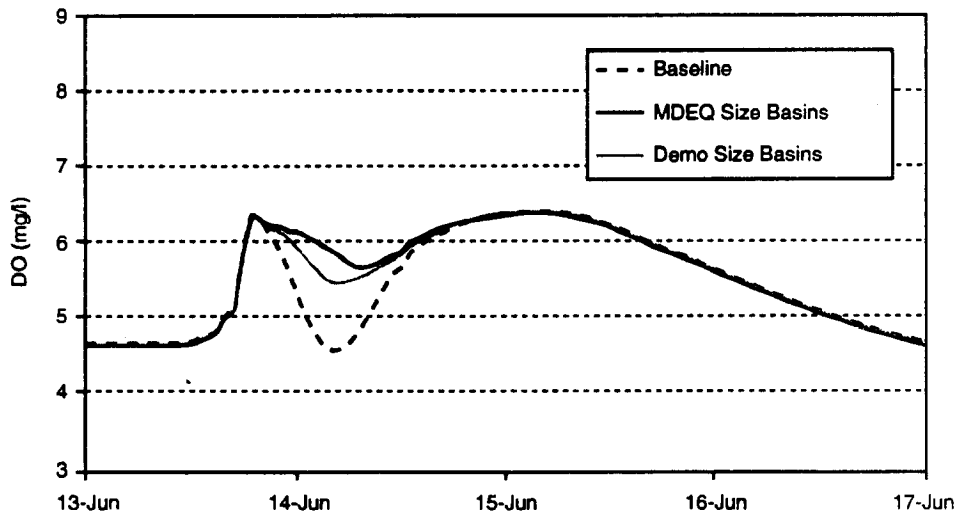

Figure 5. Modeled DO for CSO control alternatives - Main Rouge at 8 Mile Rd.

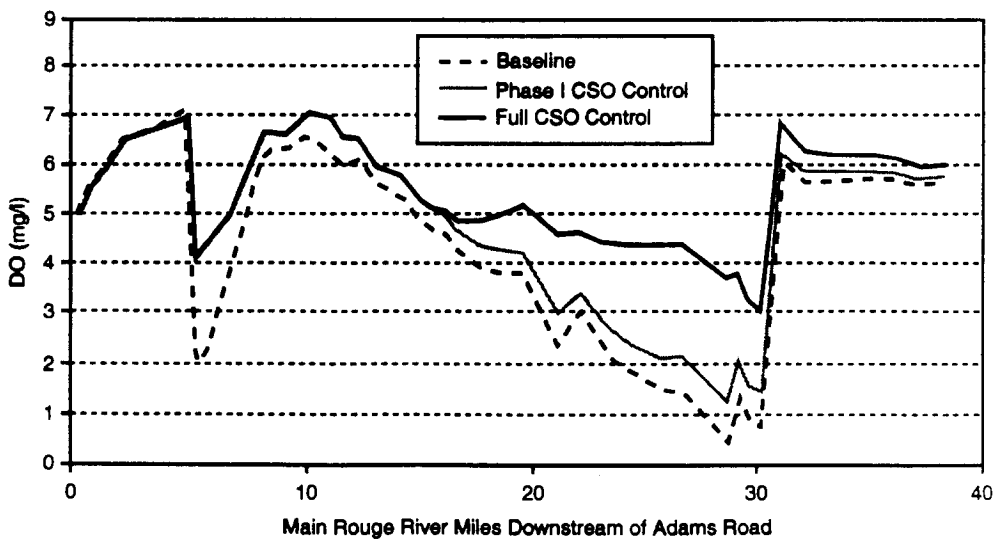

Figure 6. Dry weather model instream DO for June 13, 1994 - Main Rouge from Adams to Greenfield.

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- In a predominantly urban watershed, increased stream flow due to urbanization damages aquatic habitat and causes bank erosion, log jams, sedimentation and increased instream solids concentrations.

- Some dry and wet weather DO impairment is expected to remain in the Rouge Watershed after all CSO are controlled, simply due to nonpoint stormwater runoff. Stormwater and CSO must be addressed together in a holistic approach.

- It is expected that the Rouge Watershed CSO basins sized to demonstration criteria will be adequate to eliminate any resultant water quality impairments.

- Rouge Watershed standard practices for on-site detention of stormwater do little to mitigate the development-induced flow increases for small storms, with attendant increased velocities and streambank erosion.

Lessons Learned

Over the course of the Rouge Project modeling effort a number of lessons have been learned about the modeling. Several of the key lessons are:

- If possible, model selection and development should not be performed until the specific questions to be addressed by the model are well formulated.

- A simple loadings model such as WMM can be a good technical resource, but it may be even more important as a tool for communicating technical findings.

- Urban rivers dominated by stormwater runoff present a unique modeling challenge as the difficulty of monitoring nonpoint sources means there are not well-defined inputs for the model.

- Models should not be used to try to answer every question. Many questions can still be answered via analysis of monitoring data.

Conclusions

The Rouge project is successfully using a suite of four water quality modeling tools to address technical questions raised in watershed management planning. The Rouge Project models, modeling approach and findings are a resource that is transferable to other urban watersheds.

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Overview of Urban Retrofit Opportunities in Florida

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Abstract

With the implementation of Florida's State Stormwater Rule in February, 1982, stormwater discharges serving new development or redevelopment were required to be treated by the incorporation of site appropriate best management practices (BMPs) into the project's stormwater management system. The implementation of this program has greatly reduced the impact of stormwater discharges on aquatic resources, especially given Florida's rapid growth which has seen the state's population grow from 9,746,224 in 1980 to an estimated 14,700,000 in 1997. However, stormwater discharges from development existing before 1982 continue to contribute to the degradation of Florida's water resources. This paper will review the institutional framework the state has implemented to address stormwater problems associated with existing land uses. Its primary focus will be to summarize several different types of urban stormwater retrofitting projects that have been undertaken to reduce pollution from older stormwater discharges. For each project, we will review the type and design of BMP, site characteristics, cost, and pollutant removal efficiency.

Introduction

Florida is blessed with a multitude of natural systems, from the longleaf pine-wiregrass hills of the panhandle, to the sinkhole and sand ridge lakes of the central ridge, to the Everglades "River of Grass", to the coral reefs of the Keys. Abundant surface water resources include over 20 major rivers and estuaries along with nearly 8,000 lakes. Plentiful groundwater aquifers provide over 90% of the state's residents with drinking water. Add the state's climate and it is easy to see why many consider the Sunshine State a favored vacation destination and why the state has experienced phenomenal growth since the 1970s. Today, Florida is the fourth most populous state and is still growing rapidly, although not at the rate of 900 people per day (300,000 per year) that occurred throughout the 1970s and 1980s.

Florida's natural systems, especially its surface and groundwater resources, are extremely vulnerable and easily damaged. This is partially the result of the state's sandy porous soils, karst geology, and abundant rainfall. The negative impacts of unplanned growth were seen as early as the 1930s, when southeast Florida's coastal water supply was threatened by saltwater intrusion into the fragile freshwater aquifer that supplied most of the potable water for the rapidly expanding population. By the 1970s, it was becoming all too clear that unplanned land use, development, and water use decisions were altering the state in a manner that, if left unchecked, could lead to profound, irretrievable loss of the natural beauty that brought residents and tourists to Florida. Extensive destruction of wetlands, bulldozing beach and dune systems, continued saltwater intrusion into freshwater aquifers, creation of impervious surfaces and resulting increase in stormwater, and the extensive pollution of the state's rivers, lakes, and estuaries were only some of the negative impacts of this rapid growth.

Fortunately, Florida's citizens and elected officials became educated about these problems and began developing programs to protect and manage the state's natural resources. Florida began serious and comprehensive efforts to manage its land and water resources and growth coincident with the increasing strength of the environmental movement in the nation and the state during the early 1970s. Over the next 25 years, Florida's natural resources management programs have evolved substantially. Collectively, the individual laws and programs enacted during this period can be considered "Florida's Watershed Management Program." In many cases, these laws have been integrated either statutorily, with revisions to existing laws, or through the adoption of regulations by various state, regional, or local agencies. A primary focal point has been the management of nonpoint sources of pollution, especially urban stormwater, since stormwater discharges are responsible for over half of the pollution load entering Florida's rivers, lakes, and estuaries (Camp et al., 1995)

Florida's Stormwater Treatment Program

Research conducted in Florida during the late 1970s characterized stormwater pollutants, provided cost and benefit information on many types of stormwater treatment

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practices, and determined the importance of stormwater discharges as a major source of pollution. As a result, in 1979, the Florida Environmental Regulation Commission adopted the state's first stormwater treatment requirements. In 1982, the state's stormwater rule was fully adopted, requiring all new development and redevelopment projects to include site appropriate BMPs to treat stormwater. This technology-based program establishes a performance standard of removing at least 80% of the average annual post-development loading of total suspended solids for stormwater discharged to most waters. Stormwater discharges to the state's most pristine waters, known as Outstanding Florida Waters, are required to reduce pollutant loading by 95%. As a result of the implementation of Florida's stormwater treatment program, the impact of the state's rapid growth on its water bodies has been greatly reduced.

With the successful implementation of Florida's stormwater treatment, wetlands protection, and growth management programs to address the adverse impacts of new development, the focus of Florida's watershed management program has shifted to cleaning up "older sources," such as existing urban or agricultural land uses, and to integrating program components to eliminate duplication and improve efficiency and effectiveness. This has led to greater emphasis on more holistic approaches to address cumulative effects of land use activities within a watershed and to a greater emphasis on regional structural controls and the purchase or restoration of environmentally sensitive lands. Key institutional aspects of this changing focus include:

1985 Chapter 403.0893, F.S., is created as the only surviving section of a stormwater management bill that was developed over a ten month process. The bill was an attempt to put into law a cost-effective, timely process to retrofit existing drainage systems to reduce the pollutant loadings discharged to water bodies. The only section enacted creates explicit legislative authority for local governments to establish stormwater utilities or special stormwater management benefit areas. Today, over 80 Florida local governments have implemented a stormwater utility to provide their stormwater programs with a dedicated source of funding.

1987 Chapter 373, F.S., is revised to add a new section, the Surface Water Improvement and Management (SWIM) Act, which establishes six state priority water bodies. It directs the state's five regional water management districts (WMDs) to prepare a priority water body list and develop and adopt comprehensive watershed management plans to preserve or restore these water bodies. The bill provides $15 million from general revenue sources and requires a match from the WMDs. Unfortunately, a dedicated funding source is not established making the program dependent upon uncertain annual legislative appropriations.

1988 The State Nonpoint Source Assessment and Management Plan, prepared pursuant to Section 319 of the Federal Clean Water Act, is submitted to EPA and approved. This qualifies the state for Section 319 NPS Implementation grants which are used for BMP demonstration projects and to refine existing NPS management programs. The delineation of the state's ecoregions, selection of riverine ecoregion reference sites, and modification of EPA's Rapid Bioassessment Protocols and metrics for use in Florida is initiated. This will provide the state with better tools to assess cumulative impacts of stormwater discharges and the effectiveness of stormwater management efforts.

1989 Chapters 373 and 403, F.S., are revised as part of the 1989 Stormwater Bill. The legislation clarifies the stormwater program's multiple goals and objectives; sets forth the program's institutional framework which involves a partnership among DER, the WMDs, and local governments; defines the responsibilities of each entity; addresses the need for the treatment of agricultural runoff by amending Chapter 187, F.S., to add a policy in the Agriculture Element to "eliminate the discharge of inadequately treated agricultural wastewater and stormwater;" further promotes the watershed approach used by the SWIM Program; designates State Water Policy, an existing but little used DER rule, as the primary implementation guidance document for stormwater and all water resources management programs; and creates the State Stormwater Demonstration Grant Program with $2 million in funding as an incentive to local governments to implement stormwater utilities.

1990 Chapter 17-40, FAC, State Water Policy undergoes a total revision and reorganization so that it can be used as guidance by all entities implementing water resource management programs and regulations. Section 17-40.420 (now 62-40.432) is created and includes the goals, policies, and institutional framework for the state's stormwater management program. Key elements are:

- DER is designated as the lead agency with responsibility for setting program goals, providing overall program guidance, overseeing implementation of the program by the WMDs, and coordinating with EPA, especially with the advent of the new NPDES stormwater permitting program.

- WMDs are the chief administrators of the stormwater regulatory program (quantity and quality). They are responsible for preparing SWIM Watershed Management Plans, which include the establishment of stormwater pollutant load reduction goals (PLRGs), and for providing technical assistance to local governments, especially with respect to basin planning and the development of stormwater master plans.

- Local governments are the front lines in the stormwater/watershed management program since they determine land use and provide stormwater and other infrastructure. They are encouraged, but not required, to set up stormwater utilities to provide a dedicated funding source for their stormwater program. Their stormwater responsibilities include preparation of a stormwater master plan to address needs imposed by existing land

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uses and by future growth; operation and maintenance activities; capital improvements of infrastructure; and public education. They are encouraged to set up an operating permit system wherein stormwater systems are inspected annually to ensure that needed maintenance is performed.

Important stormwater program goals include:

- Preventing stormwater problems from new land use changes and restoring degraded water bodies by reducing the pollution contributions from older stormwater systems.
- Retaining sediment on-site during construction.
- Trying to assure that the stormwater peak discharge rate, volume, and pollutant loading are no greater after a site is developed than before.
- An 80% average annual load reduction for new stormwater discharges to most water bodies.
- A 95% average annual load reduction for new stormwater discharges to Outstanding Florida Waters.
- Reducing, on a watershed basis, the pollutant loading from older stormwater systems as needed to protect, maintain, or restore the beneficial uses of the receiving water body. The amount of needed pollutant load reduction is known as a "Pollutant Load Reduction Goal or PLRG."

With the inclusion of the PLRG concept in State Water Policy, Florida has an institutional framework to begin focusing efforts on the reduction of environmental impacts from existing land uses. While the focus of this paper is on urban stormwater, many projects to reduce stormwater pollution from agricultural activities have been undertaken, including the construction of tens of thousands of acres of wetlands to help restore the Everglades.

The rest of this paper will include short summaries of successful urban stormwater retrofitting projects that have been undertaken in Florida. These are representative of the different types of structural BMPs that are being used to reduce the impacts of urban stormwater discharges on state's waters. However, it is also important to remember that nonstructural pollution prevention programs are also a crucial element of urban retrofitting. Educational efforts, whether signage associated with a structural retrofit project or statewide efforts such as "Florida Yards and Neighborhoods," are essential in reducing "pointless personal pollution" and in gaining the support of citizens and elected officials for stormwater management programs.

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Urban Stormwater Retrofitting Project Fact Sheet

Lake Jackson Megginnis Arm Regional Stormwater System
Northwest Florida Water Management District

Watershed Area: 2200 acres

Watershed Land Use:
1191 acres - Low-medium density Residential
102 acres - High density Residential
469 acres - Commercial
213 acres - Roads
207 acres - Open

Project Overview: Studies in the mid-1970s of Lake Jackson in Leon County, Florida, determined that stormwater from the rapidly urbanizing Megginnis Arm watershed and from the construction of Interstate 10 were responsible for the lake's water quality degradation. In 1983, the NWFWMD and the FDER cooperatively designed and constructed, using EPA Clean Lakes grant and state funds, an experimental regional stormwater treatment system. The system consists of a 20 acre wet detention pond with a heavy sediment basin at the inflow, a 4.2 acre sand filter system, and a 5.7 acre, three cell constructed wetland. The pond originally was sized for 150 acre-feet of storage, representing the runoff from a 2.5 inch storm in the watershed. Continued urbanization of the watershed resulted in greater volumes of stormwater, thereby reducing the system's effectiveness. Therefore, the system was enlarged in 1989-90 to increase the storage volume by 31.7% thus providing 173.8 acre-feet of storage, or storage for 1.02 inches of runoff from the watershed. In 1992, the sand filter system was completely renovated, including new distribution pipes and sand filter media. Finally, in 1990-92, over 112,000 cubic yards of sediments which had accumulated in the bottom of Megginnis Arm were removed and the littoral areas of the arm were replanted with native macrophytes and trees.

Project Cost:

Original construction - $2,664,389	Filter renovation - $80,000
Pond expansion - $253,643	Dredging Megginnis Arm - $990,311
Educational signs - $19,565	Educational program - $40,000

Educational Component: Educational exhibits were installed at five public boat landings on Lake Jackson to increase public awareness about stormwater pollution, the regional stormwater treatment system, and the dredging project. The NWFWMD created the "Teacher's Guide to Stormwater Runoff in the Lake Jackson Watershed" and a video entitled "In Search of Old Bigmouth" as resource materials for local teachers. These are used in conjunction with a field trip program for local schools which provides students with "hands on" experience in water quality monitoring and the operation of the regional stormwater treatment system. More than 3,000 students have participated in this program.

Project Evaluation: About 6,000 cubic yards of materials were dredged from the heavy sediment basin after three years of operation with additional material removed during the system's expansion. Monitoring data shows that in normal operation, the system can reduce total volume by 30% and reduce loadings by over 90% for solids, 70% for total nitrogen, 80% for total phosphorus, and 50% for orthophosphorus.

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Urban Stormwater Retrofitting Project Fact Sheet

Lake Ella Alum Injection System
City of Tallahassee Stormwater Utility

Watershed Area: 157 acres

Watershed Land Use:

13 acres - Residential	15 acres - Open
115 acres - Commercial/Residential	3 acres - Church
11 acres - Commercial	1 acres - Street

Project Overview: In 1985, a lake restoration project was initiated in Lake Ella, a shallow, 13.3 acre hypereutrophic "lake" which receives stormwater runoff from a 157 acre highly impervious watershed. Due to its highly developed and urban watershed, and because of the low permeability of the watershed's clay soils, it was determined that traditional stormwater treatment BMPs could not be used. Instead, chemical treatment of runoff was evaluated using various chemical coagulants including aluminum sulfate (alum), ferric salts, and polymers. Jar tests determined that alum consistently provided the highest removal efficiencies and produced the most stable end product. Consequently, a prototype alum injection system was designed where liquid alum was injected within storm sewers on a flow weighted basis. Standard triplex metering pumps are used as the injection pumps, each individually regulated by sonic flow meters attached to the storm sewer lines to be treated. Many of the smaller storm sewers were combined to reduce the points of discharge into the lake from 17 to ten. Six of these ten inputs, representing 95% of the average flow, are equipped with alum injectors. Alum is pumped from a 6000 gallon alum storage tank into individual one inch PVC underground carrier lines to the point of injection. The alum mixes with stormwater as it travels through the storm sewers, passes through a fine mesh trash trap, and is discharged into Lake Ella. The restoration project also included the removal of 50,000 yds^3 of accumulated sand, debris, and muck from the bottom of Lake Ella and the recontouring of the lake's bottom with a gradual slope toward the outfall control structure.

Project Cost: The city's stormwater utility paid $744,000 for the Lake Ella restoration project, with the alum system costing $200,400. At a cost of $137/dry ton of liquid alum, annual chemical costs for alum injection are approximately $10,000 per year.

Project Evaluation: Pre- and post-alum injection monitoring is summarized below:

Parameter	Before	After	Parameter	Before	After
pH	7.41	6.43	DO	3.5 mg/l	7.4 mg/l
Total Nitrogen	1876 ug/l	417 ug/l	Total Phosphorus	232 ug/l	26 ug/l
BOD	41 mg/l	3.0 mg/l	Chlorophyll-a	180 mg/m^3	5.1 mg/m^3
Secchi Depth	0.5 m	2.2 m	Florida TSI	98	47

Alum sludge accumulation rate: 0.33 cm/yr
Pollutants in sediments are much more tightly bound after alum injection system.

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Urban Stormwater Retrofitting Project Fact Sheet

Project Smart - Stormwater Reuse Demonstration
City of Winter Park and the University of Central Florida

Watershed Area: 8.13 acres

Watershed Land Use: 6.84 acres Impervious Residential/Commercial
1.29 acres Greenspace
84% impervious with 42% DCIA

Project Overview: Lake Mendsen is a small urban constructed pond which has been altered significantly over many years and also receives untreated urban stormwater runoff. The primary discharge from the pond occurs to two drainage wells. The demonstration project was implemented to try to reduce the amount of untreated stormwater which is discharged to the pond and ultimately the Floridan Aquifer by detaining a portion of the first flush of stormwater so that it can be used for irrigation purposes or "reuse."

An area of the pond (approximately 0.7 acres) which receives stormwater from two existing outfalls was isolated from the main pond by the construction of a berm and weir system. The isolated area serves as a surface water reservoir for the irrigation system. Accumulated sediments and invasive exotic vegetation also were removed from the area and the bottom was recontoured. The resulting littoral zone was planted with five species of native aquatic macrophytes. Instrumentation was installed to monitor rainfall, irrigation pumping rates and volumes, and discharge volumes from the reservoir to the main body of the pond.

Project Cost: The entire project cost $143,000, although capital costs for the irrigation pump and system was only about $4,600. Funding for the project was provided by the DER Pollution Recovery Trust Fund ($79,000) and by the city of Winter Park and the University of Central Florida which provided $64,000 in money and in-kind services.

Project Evaluation: A mass balance was performed for the reuse pond over a study period of 358 days. The average irrigation rate for the study period was approximately 1.07 inches per week over the 1.25 acre greenspace. The overall mass balance demonstrated that 55% of the incoming runoff was reused and not discharged into Lake Mendsen. Based on Florida rainfall statistics and stormwater characterization data, this translates into an annual stormwater pollutant load reduction of over 80% for all pollutants. The project also resulted in a real economic benefit. Annualized cost savings for irrigation were calculated to be approximately $3,300 per year, based on the reuse of stormwater versus the use of potable water from the city of Winter Park.

R0022499

Urban Stormwater Retrofitting Project Fact Sheet

Lake Greenwood Urban Wetland
City of Orlando Stormwater Utility

Watershed Area: 527 acres

Watershed Land Use: 275 acres - Residential; 210 acres - Commercial/Industrial; 28 acres - Open; 14 acres - Water

Project Overview: The Greenwood Urban Wetland was built to alleviate flooding and to treat stormwater runoff prior to discharge to drainage wells which flow to the Floridan Aquifer. The system is designed to detain the runoff from 2.5 inches of rainfall. Approximately 300,000 cubic yards of material was removed to create the system which enlarged the surface area of the "lake" from four to thirteen acres. Weirs were constructed to control water levels and establish three ponds to maximize stormwater detention. The average water depth is 5.1 feet, the storage volume is 66 acre feet, and the hydraulic residence time is 22.7 days. The lakes have a 25 to 30-foot-wide littoral shelf which was planted with over 82,000 plants of ten species of native macrophytes. The lakes are connected by marsh flowways and the system also includes a "riverine floodway" that allows large storms to bypass the lake system. The floodway is planted with seven species of hardwood swamp trees. An upstream sediment/debris basin, pond aeration, and an irrigation system reusing stormwater are incorporated into the design to increase pollutant removal effectiveness. The reuse system allows the City to irrigate the park and the adjacent city-owned cemetery with stormwater instead of potable water, saving the city $25,000 per year. In addition to providing flood protection and stormwater treatment, the 26 acre Lake Greenwood Urban Wetland park includes sidewalks, bridges, and green space passive recreation which is widely used by nearby residents.

Project Cost: $581,000 from the City of Orlando Stormwater Utility.

Project Evaluation: Preconstruction monitoring was conducted from May 19, 1987 through October 13, 1988 to determine the trophic state of Lake Greenwood and to determine the potential loadings discharged to the lake's five drainage wells. The preconstruction Trophic State Index averaged 64 and was highly variable ranging from 12. 5 to 80.8 with five months above 70. After construction, TSI values averaged 57 but no months had values above 66 and variability was less with a range of 36.2 to 66.3. Treatment effectiveness of the system is summarized below:

	TN	NO2	NO3	NH4	TP	OP	Cd	Cu	Pb	Zn
Sed Trap	4%	-76%	4%	-100%	11%	7%	26%	19%	10%	6%
Wetland	11%	8%	-13%	10%	62%	77%	0%	59%	60%	69%

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Urban Stormwater Retrofitting Project Fact Sheet

Packed Bed Wetland Filter System
City of Orlando Stormwater Utility

Watershed Area: 121 acres

Watershed Land Use: 75 acres - Commercial
14 acres - Stadium/parking
8 acres - Industrial
18 acres - Roads
6 acres - Open space

Project Overview: Clear Lake is 360 acres in size and stormwater loadings from its three square mile watershed have led to serious water quality problems. An innovative stormwater treatment system was needed for this basin to both reduce pollutant load and function within a limited area where multiple demands are placed on the use of land. The constructed experimental stormwater treatment train consists of:

- A 3.3 acre off-line wet detention pond with a sediment trap at the inlet.
- Construction of diversion weirs to shunt the first flush to the wet detention pond while allowing the remaining stormwater to bypass the system.
- Construction of 10 packed beds consisting of five crushed concrete and five granite media beds, vegetated with four differing combinations of wetland plants.
- Installation of two pumps to supply water to the packed beds from both the wet detention system during storms and from Clear Lake during dry periods.
- Control valving to allow for varied water flow rates through the packed beds.
- Automated flow meters and composite samplers to allow storm event sampling.

Project Cost: $917,464 including monitoring costs with funding from DEP through the State Stormwater Demonstration Grant Program and from the City's stormwater utility.

Project Evaluation: Monitoring was performed on the effectiveness of the overall system, the performance of the individual beds, and the best flow rate at which to operate the system (30, 60, or 120 gal/min). Analysis of the individual beds showed consistent removal across all beds for cadmium, copper, lead, zinc, total nitrogen, TKN, nitrite, total phosphorus, TSS, VSS, and fecal coliform. Among the remaining parameters, chromium, ammonia, nitrate, orthophosphorus, TDS, and TOC, pollutant removals within bed 6 were consistently low at all three flow rates. Conversely, bed 5 exhibited consistently high removals for the same parameters. The high flow rate was determined to be the best operating rate for the system. Overall pollutant load reduction is presented below:

Parameter	% Removal	Parameter	% Removal	Parameter	% Removal
Cadmium	80	Total Nitrogen	63	Total phosphorus	82
Chromium	38	TKN	62	Orthophosphorus	14
Copper	21	Ammonia	6	TDS	8
Lead	73	Nitrate	75	TSS	81
Zinc	55	Nitrite	-9	VSS	80
Fecal Coliform	78			TOC	38

R0022501

Urban Stormwater Retrofitting Project Fact Sheet

Bath Club Concourse Stormwater Rehabilitation Project
Town of North Redington Beach, Pinellas County, Florida

Watershed Area: 2.12 acres

Watershed Land Use: Pre-project - 100% Impervious Roadway/Parking

Project Overview: The Bath Club Concourse is a combination roadway and parking lot connecting Bath Club Circle and Gulf Boulevard. Before the project, the Bath Club Concourse was totally impervious consisting of asphaltic pavement. Untreated runoff from the Concourse and its associated drainage area was directed by sheet flow into a single storm sewer inlet and discharged offsite, and ultimately to Boca Ciega Bay.

The objectives of this project were: (1) to maximize the amount of stormwater runoff that could be infiltrated on-site, thereby reducing the annual volume that is discharged off-site without any treatment; and (2) to demonstrate innovative alternative approaches to treating stormwater in highly urbanized areas where land for traditional BMPs is scarce and very expensive. Drainage was redirected toward two new pervious concrete parking areas located in the center of the Concourse. These are separated by an unpaved landscaping island that also provides infiltration. To maximize infiltration of the pervious concrete parking areas, two 150-feet-long underdrains were installed in the eastern half of the project to facilitate the drainage of the subsurface soils immediately beneath the pervious concrete.

Project Cost: Total cost was $147,015 with construction costing $118,380 and landscaping costing $13,345. Funding was provided by a Section 319 NPS grant from DER, the SWFWMD SWIM Program, and the Town of North Redington Beach.

Project Evaluation: The project improvements resulted in a significant reduction of direct discharge of stormwater runoff from the site. Calculations accounting for average annual rainfall and runoff, as well as pore space volume and subsurface water flow, indicate that the improvements caused a 33% reduction in total on-site runoff volume between the pre- and post-project conditions. Further, the volume of surface runoff discharging directly to Boca Ciega Bay was reduced by about 75%. Calculated overall removal efficiencies for the project are based on the efficiency of the underdrain/filter system to remove pollutants and are indicated as follows:

Parameter	Lead	Zinc	TSS	BOD	TP	OrthoP	TN
% Removal	73	72	73	61	49	26	65

R0022502

Urban Stormwater Retrofitting Project Fact Sheet

Sunset Drive Outfall Stormwater Rehabilitation Project
City of South Pasadena, Pinellas County, Florida

Watershed Area: 49 acres

Watershed Land Use: 21.6 acres - Residential Multifamily
20.1 acres - Commercial
7.4 acres - Residential Single Family

Project Overview: The Sunset Drive drainage basin is nearly fully developed and consists of approximately 55% impervious area. Historically, stormwater was collected and discharged untreated to a local storm sewer which connects to a City of St. Petersburg storm sewer main. This storm sewer main ultimately discharges to Boca Ciega Bay.

The objectives of this project were: (1) to reduce stormwater pollutant loading to Boca Ciega Bay by incorporating an in-line sediment sump/oil and grease skimmer in the Sunset Drive storm sewer system before its junction with the larger storm sewer main; and (2) to demonstrate innovative alternative approaches to treating stormwater in highly urbanized areas where land for traditional BMPs is scarce and very expensive. The sump was designed, to the extent possible, to meet the current rule requirements for this type of system. Due to physical limitations, the design provided for the storm sewer flow to be diverted to the area of an existing greenspace for treatment, prior to being diverted back to the main flow path of the storm sewer. The greenspace, which is adjacent to the bay, was modified into an open, linear wet-sump, which included energy dissipaters and a skimmer baffle. The project also included an attractive boardwalk around and over the facility as well as plantings of salt marsh vegetation in the sump's littoral zone.

Educational Component: The architecture and location of the boardwalk serves to attract pedestrian traffic to the project. Being located immediately in front of City Hall provides an excellent high-profile example of how local government can cooperatively implement measures to reduce stormwater pollution. Several interpretive signs provide information regarding nearshore aquatic plants and animals and the value of stormwater treatment.

Project Cost: Total cost was $115,000 with construction costing $83,131. Funding was provided by a Section 319 NPS Grant from DER, the SWFWMD SWIM Program, and the City of South Pasadena. A grant from the Tampa Bay National Estuary Program paid for the educational signs.

Project Evaluation: The project provides an opportunity to trap and retain sediment and other suspended materials as small as 0.1 mm in diameter. A corresponding reduction in other urban pollutants typically associated with suspended solids such as heavy metals, bacteria, and oxygen demanding substances can also be expected. The sediment load reduction to Boca Ciega Bay is estimated to be approximately 24.5 cubic yards per year.

R0022503

Urban Stormwater Retrofitting Project Fact Sheet

EMS Stormwater Enhancement Project
Pinellas County, Florida

Watershed Area: 9.24 acres

Watershed Land Use: 9.24 acres - Mixed Use (85% Impervious)

Project Overview: The original stormwater facility was constructed in accordance with regulations in 1990 to provide stormwater treatment and peak attenuation for the county's new Emergency Medical Services (EMS) complex. The facility discharges indirectly into Boca Ciega Bay. The pond was designed to capture stormwater and treat, using a sand filter encased in a concrete vault, the first half-inch of runoff from the entire site. The facility was constructed with 4:1 side slopes, 2 foot average water depth, and a 0.4 foot treatment prism for capturing and filtering runoff. Prior to the enhancements, a monoculture of primrose willow dominated the entire perimeter of the pond.

The primary objective of this project was to demonstrate how stormwater ponds can be designed to enhance their aesthetic and wildlife habitat values while at the same time meeting their intended water quality treatment and/or flood control purposes. The secondary objective was to actually improve the treatment effectiveness of the existing pond by expanding and planting the pond's littoral zone, increasing the treatment volume between the control elevation and overflow weir, and increasing the permanent pool volume, thereby increasing the residence time in the pond.

Educational Component: Due to the adjacent location of the County's Cooperative Extension Service, the project is readily available for touring by anyone visiting the Extension Service. Educational display boxes at various locations along the mulched path surrounding the pond provide information regarding the importance and function of stormwater treatment facilities. Also, as part of the project, the Extension Service produced a 28 minute educational video entitled "Stormwater Ponds: The new Urban Wetlands." While the video discusses the importance of treating stormwater, it focuses primarily on the potential value of stormwater ponds for providing improved urban wildlife habitat. The video is used to inform groups such as homeowner associations, condominium associations, and civic associations, about stormwater pollution and management.

Project Cost: Total cost was $78,500, with construction costing $63,244. Funding was provided by a Section 319 NPS Grant from DER, the SWFWMD SWIM Program, and Pinellas County.

Project Evaluation: By more than doubling the permanent pool volume of the pond, the pond's residence time was substantially increased. The pond's treatment volume also was increased by 13.4%, from 0.50 inches of runoff to 0.57 inches. The increased residence time allows for longer periods of physical settling as well as biological activity. The reshaping and replanting of the littoral shelf resulted in increased nutrient uptake.

R0022504

Urban Stormwater Retrofitting Project Fact Sheet

Jungle Lake Water Quality and Habitat Enhancement
Southwest Florida Water Management District

Watershed Area: 1000 acres

Watershed Land Use:

Project Overview: Walter Fuller Park is a highly used recreational/athletic park located in the western part of the city of St. Petersburg, approximately 2.5 miles east of Boca Ciega Bay. Jungle Lake was excavated about 75 years ago to provide fill for the construction of local roads. The 11.2 acre kidney-shaped lake received untreated stormwater from five inflows and discharges to the bay via a single outflow. During most storms, runoff bypassed Jungle Lake and was discharged directly to the bay. To improve the quality of water in the lake and that which is discharged to the bay, a BMP treatment train was constructed. The system includes:

- A diversion weir so that most stormwater is routed into the lake for treatment instead of directly into the bay.
- Modification of the inflow ditches to create shallow sloughs vegetated with native aquatic macrophytes.
- Expansion of the lake to create littoral zones vegetated with macrophytes.
- Two partially submerged berms which produce a longer flow path, increase residence time, provide natural habitat, and replace park uplands resulting from the lake perimeter modifications.
- Sediment sumps at the northeastern and southeastern inflows.
- An oil and grease skimmer on the outfall structure.
- Over 15,000 herbaceous plants consisting of 11 species, 170 trees, and 700 shrubs.

Project Cost: $328,000, which included $59,000 from the City of St. Petersburg and $269,000 from the SWFWMD SWIM Program. About 51,000 cubic yards of fill were needed for the project, of which over half would need to be imported at a cost of $3.80 to $4.75 a cubic yard. Since the area northeast of the lake was three feet higher than the surrounding roads and fields, this area was excavated instead. Within this area, two soccer fields were designed and constructed to provide the community with additional recreational facilities and to promote park usage. Even after the sodding of the soccer fields and the installation of an irrigation system, over $35,000 was saved.

Educational Component: During the conceptual planning and design phases, the City and SWIM staff met with members of the Jungle Lake Civic Association to obtain their input and to extend their ownership of the park to include the stormwater improvements. The members assisted in the selection of plants and received a grant to supplement the wetland and forest plantings. They also assisted in planting the vegetation and are participating in the educational, maintenance, and monitoring aspects of the project. The site has on display eight educational displays that inform the general public and students about stormwater issues and management. A teacher's manual was produced that can be used in the classroom or to accompany the signs during school field trips.

R0022505

Urban Stormwater Retrofitting Project Fact Sheet

Oleander Avenue Stormwater Exfiltration Trench System
City of Daytona Beach, Florida

Watershed Area: 49 acres

Watershed Land Use: Single Family Residential -
23% Directly Connected Impervious Area (DCIA)

Project Overview: The Oleander Avenue watershed historically discharged untreated runoff to storm sewers that ultimately discharged to the Halifax River. The area was also subject to periodic local flooding due to the inadequate capacity of the conveyances. The primary objective of this project is to demonstrate the cost-effectiveness of using exfiltration systems as a method of retrofitting stormwater problem areas for future use within the city's beachside community.

To alleviate the flooding problem and to reduce pollutant loading to the river, a perforated pipe exfiltration trench treatment system was constructed. Site constraints limited the treatment volume to 0.75 inches over the DCIA which translates into a storage volume of 30,700 cubic feet. The 294 feet of exfiltration system is designed to accept the runoff from a 5 year, 24 hour storm representing flows of from 1.5 to 17.5 cfs from the drainage area subbasins. Actual pipe sizes varied from 19" x 30" to 29" x 45" to meet the design storm flow conditions. The rock filled trench measures 16 feet in width and 2 feet in depth.

Project Cost: Total cost was $513,700 with construction costing $375,617. This represents a cost of approximately $10,200 per acre. Funding was provided by DEP from a State Stormwater Demonstration Grant and from the City of Daytona Beach.

Project Evaluation: The exfiltration trench appears to be functioning very well as water quality monitoring efforts have failed to find any discharge from the system. Since exfiltration systems provide 100% treatment for all water which is retained and exfiltated, this system will reduce the stormwater pollutant loadings discharged to surface waters by at least 80%, since the trenches will eliminate the discharge from over 80% of the storms that occur. The project allowed the city to identify the design and construction constraints associated with this type of treatment system as well as installation costs for these systems. This knowledge will be used as the city retrofits other basins.

R0022506

Urban Stormwater Retrofitting Project Fact Sheet

Indian River Lagoon Baffle Boxes
Brevard County Surface Water Management

Project Overview: The Indian River Lagoon National Estuary Program identified stormwater discharges as the major factor in the decline in the lagoon's health. In particular, reductions in the stormwater loadings of total suspended solids, nutrients, and freshwater are needed to restore the lagoon. The county developed an innovative BMP, the baffle box, which can be installed within existing rights-of-way as a way of retrofitting stormwater discharges where land is unavailable for traditional BMPs. Baffle boxes are large sediment traps that require regular maintenance. Sediment accumulation rates vary depending on site characteristics such as drainage area, land use, soil type, slope, mowing frequency, and base flow. The boxes accumulate from 500 to 50,000 pounds per month, and requires monthly cleaning in the wet season and cleaning every two to three months in the dry season. By the end of 1997, the county had installed 31 baffle boxes, with others under construction. As part of the implementation of the Indialantic area stormwater master plan, 11 baffle boxes currently are being installed and monitored. Three different designs are being evaluated to determine their effectiveness including: (1) a two-chamber box for small pipes and drainage areas; (2) a three-chamber box for larger pipes; and (3) two boxes in series, where one box currently exists and collects large amounts of sediment.

Project Drainage Area and Cost: The average cost of installing a baffle box is around $22,000 and the average clean out cost is $450 (by private contractor). Funding from a Section 319 NPS Grant from DEP and from the County's stormwater utility are paying for the Indialantic projects. These all serve mainly residential land uses. The construction costs and watershed drainage area are summarized below:

Project	Drainage area	Cost	Project	Drainage area	Cost
Alamanda	1.8 acres	$14,376	Franklin (2)	36 acres	$33,362
Rivershore	7.2 acres	$9,463	Riverside	161 acres	$24,944
Indialantic I	25 acres	$13,580	Sunset Part	24 acres	$23,422
Monaco	54 acres	$32,835	Puesta Del	2.2 acres	$25,181
Pinetree	134 acres	$33,925	Cedar Lane	0.9 acres	$25,027

Project Evaluation: The monitoring program for the 11 new baffle boxes will not begin until the spring of 1998. However, previous assessments of the effectiveness of baffle boxes on 22 existing systems is shown below:

The county has also installed a continuous deflective separation unit, a new BMP from CDS Technologies of Australia. This unit cost $55,000 to install and treats the runoff from a 40 acre watershed. This unit captures 100% of floatables and has been cleaned out twice resulting in the removal of 8,013 pounds of sediment.

R0022507

Urban Stormwater Retrofitting Project Fact Sheet

Oil and Grease Removal BMP Demonstration
City of Oakland Park, Florida

Watershed Area: 5 acres

Watershed Land Use: Mixed commercial and industrial (95% Impervious)

Project Overview: The City of Oakland Park received one of the state's Stormwater Demonstration Grants to develop and monitor a prototype BMP for in-line removal of oil and grease from stormwater using oil absorbent material. The Northeast 40th Court site was chosen because inspection of the storm sewer system revealed substantial amounts of oil and grease. These were attributed to the large number of automobile repair shops, paint shops, plating shops, and similar businesses in the drainage area. The project consisted of characterizing the concentrations of oil and grease in the stormwater, a review of the material safety data sheets of three different oil sorbent materials, a laboratory bench scale study of one of the oil sorbent materials, construction of the BMP system, and effectiveness monitoring. The final BMP system included diversion box with a weir to direct runoff into the treatment system. As stormwater enters the treatment unit, flow is directed against an aluminum baffle imparting a slight rolling motion which causes floatables and trash to be trapped against the baffle wall for easy removal. Upon entering the treatment chamber, velocity slows greatly, allowing grit, sludge, and oil particulate matter to settle to the sloping bottom. The stormwater is then redirected upward through two cross-layers of the absorbent media, which are secured by being sandwiched between two aluminum grates, where free oil and grease are removed via absorption into the material. The absorbent media chosen was custom made by NewPig Corporation of Tipton, Pennsylvania. The product, called the Spaghetti Pillow, consists of shredded strips of polypropylene packaged in tough, UV resistant mesh skin in the shape of a rectangular bag or pillow. The two layers of media are placed perpendicular to each other to avoid short circuiting.

Project Cost: Total cost of the project was $260,870. This included $71,490 for the construction of the treatment system and $189,380 for sampling equipment, consultant, and laboratory fees.

Project Evaluation: Inflow and outflow sampling of the system was conducted for ten storms between July 1994, and April 1995. Storm event oil and grease concentrations ranged from 0 to 261 mg/l, with mean pollutant concentrations ranging from 1.41 to 85.58 mg/l. Oil and grease mass removal efficiencies ranged from 71% to 95%, while flows ranged from 0 to 1.75 cfs. The absorbtion efficiency of the filter media bags were measured twice. The amount of oil and grease absorbed ranged from 1.7 pounds to 62.5 pounds, which represents an absorbtion efficiency of 110% to 470%.

R0022508

Urban Stormwater Retrofitting Project Fact Sheet

BMP Treatment Train in the Florida Keys
City of Key Colony Beach, Florida

Watershed Area: 268 acres

Watershed Land Use:

Project Overview: Recognizing the importance of reducing stormwater pollution in protecting its sensitive natural resources, the City included in its comprehensive plan policies requiring the retrofitting of its existing drainage system. With technical assistance from the DEP and the SFWMD, the City's consultant developed a stormwater master plan in 1993. The plan included plugging 28 existing stormwater outfalls and constructing a retention basin and swales with raised inlets and exfiltration trenches which overflow into injection wells. Implementation of the master plan began in 1995, and is scheduled for completion by the year 2000. Phase 1 has been completed and Phase 2 will be completed by the fall of 1998. The stormwater master plan calls for the construction of 82,146 linear feet of swales, 9 modified raised swale inlets, about 60,000 linear feet of exfiltration trench, 35 inlet baffle systems to direct the first flush into the exfiltration trenches, and 22 injection wells.

Project Cost: The total cost of the original stormwater retrofitting master plan was estimated to be $1.2 million. However, the city's residents and elected officials decided that they did not want water standing in the swales, resulting in the addition of the exfiltration trench system. To date, using funds from the city, the DEP, and a Section 319 Grant, the city has implemented two phases of the master plan as shown below:

Basin	Acres	Swale (lf)	Sod (sF)	Exf. Trench (lf)	Injection Wells	Cost
4-1	0.66	827	29,257	445	1	$72,200
5-2	3.03	521	21,304	269	0	$47,083
2-2	3.50	1200	14,000	1200	1	$148,112
2-5	3.00	1200	14,000	1200	1	$147,790
2-8	2.13	934	11,000	934	1	$129,600
2-11	1.76	566			1	$27,854
3-1	3.69	878	11,000	878	1	$113,175
5-1	26.47	4800	37,000	3100	3	$439,773
8-2	0.02	1306	371	371	1	$72,174

Project Evaluation: Actual stormwater monitoring will not begin until the completion of Phase 2. By plugging the direct stormwater discharges to surface waters and providing storage and treatment for the first 1.5 inches of runoff, the stormwater volume and pollutant loadings will be substantially reduced. Modeling indicates that these will be reduced by up to 75% from pre-project conditions.

R0022509

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R0022510

Evaluating the Cost Effectiveness of Retrofitting an Urban Flood Control Detention Basin for Stormwater Treatment

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This paper describes the retrofitting of a flood control basin in Sunnyvale, California and subsequent monitoring to evaluate the pollutant removal effectiveness of the retrofitted basin. The authors wish to thank the Santa Clara Valley Urban Runoff Pollution Prevention Program, and especially the City of Sunnyvale for their support and cooperation in the conduct of this study.

Background

The northern portion of Santa Clara Valley experienced significant subsidence during a period of excessive groundwater pumping. In order to protect that area from flooding, a system of levees and pump stations were built. According to a survey conducted in 1990, there are 17 municipally owned and operated pump stations in Santa Clara Valley (Woodward Clyde Consulting, 1990). These pump stations generally consist of pumps, storage units such as a sump or a detention basin, and inlet and outlet works. Sumps and detention basins are designed to reduce the capacity of the pumps that would otherwise be needed to pass peak flood flows. These pump stations have generally been operated as single-purpose flood control facilities. The pump operating schedules are designed such that the pumps go on as soon as water begins to fill the basin, with the goal of emptying the basin as soon as possible after the event. These facilities were examined for their potential to provide water quality treatment in addition to flood control. One retrofitting option to achieve water quality benefits would be to change the pump operating schedule in order to increase detention time and to provide for a seasonal wet pond. Based on a preliminary evaluation of the feasibility of retrofitting detention basins, the Santa Clara Valley Urban Runoff Pollution Prevention Program decided to conduct a pilot study to retrofit a facility, and conduct testing to measure water quality benefits and costs (Woodward Clyde Consulting, 1994).

Site Description

In this pilot study, structural and operational retrofitting was conducted on Sunnyvale Pump Station No. 2 located just north of the junction of Route 237 and Calabazas Creek. The pump station consists of four primary pumps rated at 39 cfs capacity and one auxiliary electric pump (9 cfs). The detention basin area is 4.4 acres with a 30-acre-ft capacity; it receives water from a 463-acre watershed that consists of industrial park (30%), commercial (10%), and residential (60%) land uses (Figure 1).

Retrofitting Actions

The basin, originally constructed as an in-line dry detention basin with pumped outflow, was retrofitted to operate as an in-line extended detention basin with a seasonal wet pool and pumped outflow. The retrofitting required one operational and three structural changes. The detention basin has an open channel and a submerged 36-inch pipe (which was below the open channel) that connected the inlet and outlet. In order to minimize short-circuiting, a single barrier of rock was placed in the channel and a riser was placed over the entrance to the 36-inch pipe. A gabion weir was installed at the outlet to provide better distribution of flow from the basin into the outlet.

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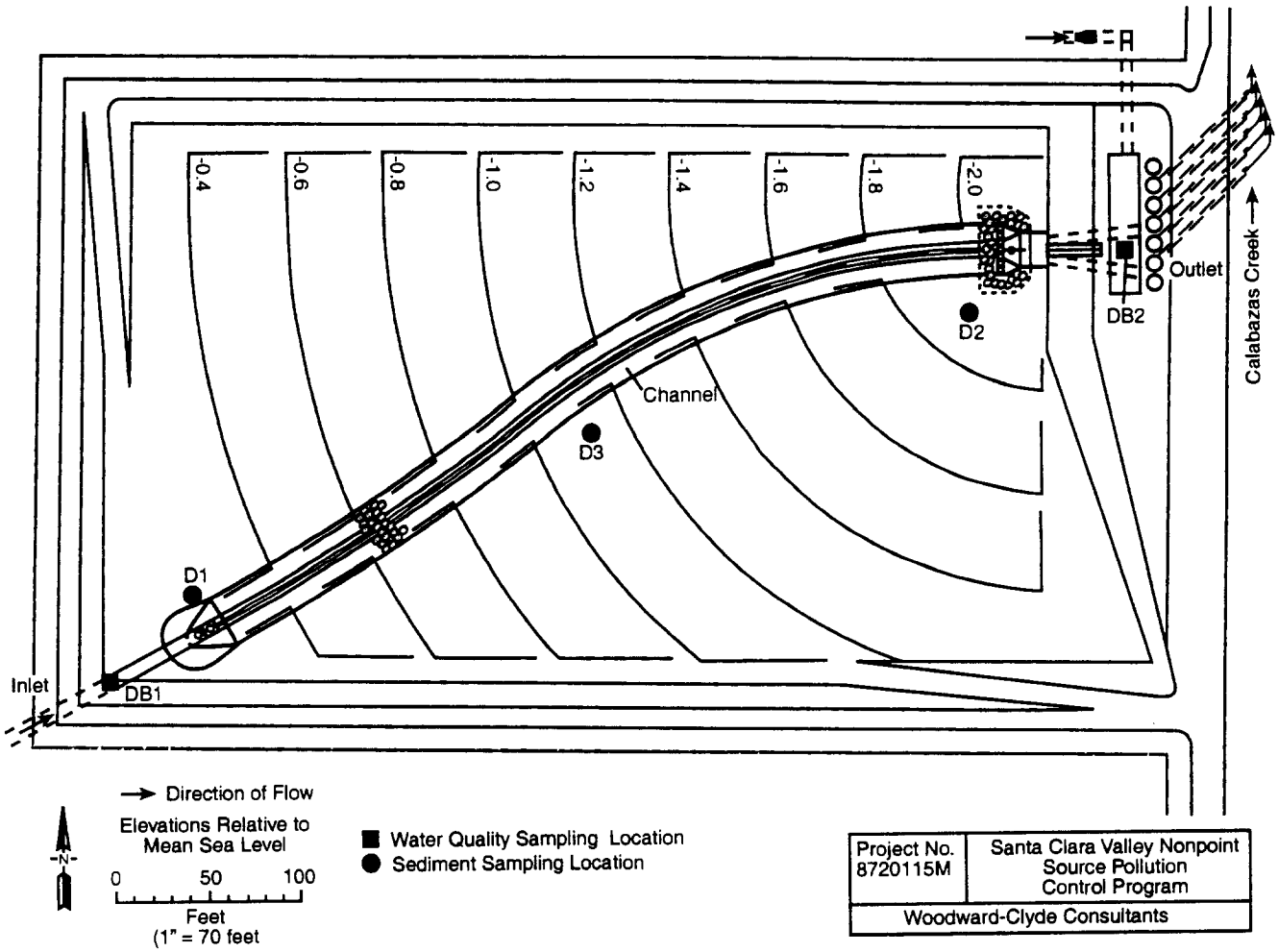

Figure 1. Sunnyvale Pump Station No. 2 showing sampling locations.

Operational changes consisted of modifying the pump schedule to create a 2-foot permanent pool at the outlet, provide a temporary pool over the depth range of 2 to 2.4 feet, and provide slow release for water depths above 2.4 feet. The 2.4-foot maximum for the temporary pool was estimated based on a flooding analysis to ensure that modifications to the pumping schedule did not cause a significant increase in the 100-year flood levels in the detention basin.

Monitoring

Flow data and water quality samples were obtained using automated samplers located in the pump station and in the pipe entering the basin. Flow-weighted composite samples were obtained from eight storm events over three wet seasons (October through April) between March 1991 and April 1993. Characteristics of the monitored storms are given in Table 1.

Water sampling was conducted for total suspended solids (TSS), selected heavy metals (total and dissolved cadmium, chromium, copper, lead, nickel, and zinc), and oil/grease. Table 2 shows how the metals concentrations at the inlet to the basin compare with data collected from other urban monitoring stations that sampled relatively single-land use catchments. Data from the detention basin inlet were most similar to residential/commercial data. Two rounds of sediment samples were also collected at three locations in the basin (near inlet, midway between inlet and outlet, and near outlet) to characterize sediment grainsizse and chemistry in the basin.

No data were obtained prior to retrofitting the facility to estimate "before and after" performance. However, based on the design and operation of the facility, pre-retrofit treatment performance was predicted to be quite low (Woodward Clyde Consulting, 1990).

Monitoring Results and Pollutant Removal Effectiveness

Because of difficulties in obtaining consistent and reliable flow measurements, pollutant removal effectiveness was estimated based on the difference between influent and effluent concentrations rather than loads (Table 3).

R0022512

Table 1. Storm Sampling Event Statistics for the Detention Basin

Storms			Rain (San Jose NWS gauge #7821)			
Sampling Event	Date	Duration (hours)	Volume (in)	Average Intensity (in/hour)	Peak Intensity in/hour)	Antecedent Dry Period (days)
SE17	3/20/91	11	0.4	0.04	0.10	2
SE20	2/12/92	17	1.3	0.08	0.20	3
SE21b[1]	3/22/92	19	0.6	0.13	0.20	1.3
SE23	12/6/92	16	1.1	0.07	0.20	39
SE24	12/10/92	6	0.9	0.15	0.30	4
SE25	1/6/93	29	0.5	0.02	0.10	5
SE27	2/17/93	60	2.2	0.04	0.20	6
SE28	3/23/93	14	1.1	0.08	0.20	7

[1] Consists of two small storms.
Note: The median event volume for the period from 1948 to 1989 was 0.49 inches.

Table 2. Comparison of Median Total Metal Concentrations in Stormwater at the Detention Basin Inlet to other Santa Clara County Monitoring Stations

	Concentrations (µg/L)			
Parameter	Detention Basin Inlet Station n = 8	Residential - Commercial Land Use Stations (L1, L3, L4, L5, L6) n = 21	Industrial Land Use Station (L2) n = 25	Open Land Use Station (L7) n = 4
Cadmium	1.1	1.0	3.9	0.3
Chromium	11.5	16.0	24.0	10.5
Copper	24.0	33.0	50.5	11.0
Lead	37.5	45.0	90.5	2.0
Nickel	20.5	30.0	46.0	5.0
Zinc	180.0	240.0	1150.0	5.0

Average removal of TSS was 50%. Removal of total chromium averaged about 30%, removal of total copper and zinc was about 40%, and removal of total lead and total nickel averaged about 50%. The data indicated that the basin did not remove dissolved metals or hydrocarbons. Estimates for removal of total cadmium were not made because the concentrations in the influent and effluent were very low (at or near the laboratory detection limit).

Sediment samples were also collected at various locations within the basin (see Figure 1) were tested for selected heavy metals (Table 4). Sediment concentrations were higher near the inlet, but none of the samples contained metals at concentrations exceeding hazardous waste criteria. Estimated sediment accumulation rates were low, as expected for a fully urbanized area, with cleanout frequencies estimated at between once every 10 or 20 years.

Cost-Effectiveness Evaluation

The amortized cost over 20 years for retrofitting, operations and maintenance (including sediment disposal) was estimated at $8,200 per year. Based on this cost and assuming flow rates were typical, the cost effectiveness of removing the metals was estimated. For example we estimated that, 1.1 lb. of copper could be removed per $1,000 spent on retrofitting. This compares well with an estimate of 1.5 lb. of copper for $1,000 spent on street sweeping.

However, the potential for significantly reducing heavy metals loads to the Bay due to retrofitting the existing flood control facilities is minimal because only a small portion of the watershed is served by detention facilities. Even if the other existing flood control facilities were retrofitted, and achieved improved removals comparable to those measured in the pilot study, the net reduction in copper would only amount to approximately 100 pounds per year, which is less than 1% of the estimated mean annual copper load to the Bay (14,000 lb.).

Conclusions

- Flood control basins, especially those with pumped outlets, may be good candidates for retrofitting for water quality control without increasing flood control risk.
- Metal removals measured in the retrofitted basin ranged between about 30-50% depending on the metal and about 50% for TSS. The basin did not appear to be effective in removing oil/grease.
- Concentrations of metals in the sediments tended to be higher near the inlet, but were well below hazardous waste criteria.
- Amortized costs for retrofitting the basin were about $8,200/year, and based on the pollutant removal per-

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Table 3. Summary of Inlet and Outlet Concentrations for Selected Pollutants at the Detention Basin

	Cadmium (µg/L)		Chromium (µg/L)		Copper (µg/L)		Lead (µg/L)		Nickel (µg/L)		Zinc (µg/L)		TSS (mg/L)	TH (mg/L)	TO&G (mg/L)
	Total	Dissolved	Total	Dissolved	Total	Dissolved	Total	Dissolved	Total	Dissolved	Total	Dissolved			
SE17															
B1 - Inlet	0.4	<0.2	3.6	1.8	8.7	5.4	6.4	2.2	1.7	<2	46	28	12	97	1.5
B2 - Outlet	0.2	<0.2	2.7	1.1	6.8	4.7	3.4	1	1.7	<2	26	19	7.3	120	1.4
Reduction	-	-	25%	-	22%	-	47%	-	0%*	-	43%	-	39%	-	7%
SE20															
B1 - Inlet	6.6	1.3	12	1	24	3	45	1	16	1	180	19	90	110	0.2
B2 - Outlet	4.8	2.5	6	1	9	3	10	1	4	1	73	22	24	63	<0.2
Reduction	-	-	50%	0	63%	-	78%	-	75%	-	59%	-	73%	-	-
SE21b															
B1 - Inlet	1.1	0.2	18	1	24	2	53	<1	25	<1	180	5	140	-	-
B2 - Outlet	1.5	<0.2	14	1	16	2	35	<1	19	<1	120	7	93	-	-
Reduction	*	-	22%	-	33%	-	34%	-	24%	-	33%	-	34%	-	-
SE23															
B1 - Inlet	1	0.2	11	<1	27	5	30	1	13	3.9	190	41	74	100	0.7
B2 - Outlet	0.6	<0.2	8.3	1.4	12	4.7	12	<1	5.8	2.2	82	45	31	90	0.5
Reduction	-	-	25%	-	56%	-	60%	-	55%	-	57%	-	58%	-	-
SE24															
B1 - Inlet	1.6	<0.2	21	1.1	40	2.1	76	<1	42	9.6	270	22	180	140	0.6
B2 - Outlet	1.3	0.2	15	8.6	24	5	40	1.4	29	15	160	31	96	140	3.5
Reduction	-	-	29%	-	40%	-	47%	-	31%	-	41%	-	47%	-	-
SE27															
B1 - Inlet	1	0.5	6.3	1.4	14	5.4	13	<1	83	63	70	35	30	110	1.6
B2 - Outlet	0.6	0.4	4.9	1.7	8.9	4.5	6.6	<1	25	20	47	26	15	220	1.3
Reduction	-	-	22%	-	36%	-	49%	-	70%	-	33%	-	50%	-	-
Average Reduction	-	-	29%	-	42%	-	53%	-	51%	-	44%	-	50%	-	-

Table 4. Sediment Grain Size and Chemistry for the Detention Basin

Station Location	Date Sampled	Type of Sample	Grainsize % Sand	Silt	Clay	% Solids	% TOC	Metals Concentrations (mg/kg) Cadmium	Chromium	Copper	Iron	Lead	Manganese	Nickel
Composite†	6/15/90	3" Core	-	-	-	-	-	2.2	-	92	-	36	-	61
D1 Inlet	5/14/92	Surficial	0	75.3	24.7	32	3.8	23	200	150	49,000	280	610	94
D3 Middle	5/14/92	Surficial	0	85.4	14.6	31	5.5	17	220	140	38,600	350	640	87
D2 Outlet	5/14/92	Surficial	0	72.4	27.6	34	1.9	35	140	47	47,700	18	680	76
D1 Inlet	7/12/93	Surficial	-	-	-	53	2.4	1.0	170	110	34,000	260	560	96
D3 Middle	7/12/93	Surficial	-	-	-	63	0.65	0.2	120	37	36,000	12	700	75
D2 Outlet	7/12/93	Surficial	-	-	-	63	0.93	0.3	110	43	30,000	24	570	73
Total Threshold Limit Concentration*								100	2500	2500	-	1000	-	2000

† Core samples were taken at station D1, D2, and D3, and composited for one analysis.
* Title 22 of the California Code of Regulations.

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formance, yielded cost-effectiveness values that were somewhat comparable to street sweeping.

- Because of the limited number of basins and the small portion of the watershed served by those basins, basin retrofitting would reduce watershed loads of metals by only about 1%.

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R0022516

Retrofitting to Protect Drinking Water Reservoirs from the Impacts of Urban Runoff

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To meet federal Surface Water Treatment Rule (SWTR) requirements and Filtration Avoidance mandates, the New York City Department of Environmental Protection has developed a proactive program to manage and protect the Kensico Reservoir and its watershed. The prime components of the program are aggressive stormwater and waterfowl management, sewer system inspection and repair, an in-reservoir turbidity curtain, reservoir dredging, and hazardous spill containment. Protecting water quality in the Kensico Reservoir is imperative because it is the final impoundment for 90% of the city's unfiltered water supply before it enters the distribution system. This paper focuses on the stormwater management element, which targets pathogens and turbidity, the key pollutants regulated by the SWTR that are conveyed to the reservoir by stormwater. The first phase of the project, watershed assessment, site selection, and stormwater management facility screening and design, is complete. Lessons learned and recommendations for planning similar efforts are summarized in this paper. Construction of the stormwater facilities is scheduled to begin in the spring of 1998 and to be completed in two phases, over a five-year period. Baseline stormwater water quality data will be collected until the facilities are constructed. Data will be collected from select stormwater facilities once they are operational in order to assess the effectiveness of the program.

Introduction

New York City has placed great emphasis on protecting and improving the quality of its drinking water supply through watershed protection and management programs. This paper describes one such program developed and implemented by the New York City Department of Environmental Protection's (NYCDEP) Bureau of Water Supply, Quality and Protection.

The city's drinking water supply system is one of the largest in the world, supplying 1.45 billion gallons of water each day to 9 million city and upstate residents. The entire watershed covers 1,969 square miles and comprises 19 reservoirs and three controlled lakes (lakes in which the city has water ownership), and numerous wetlands watercourses and intermittent streams. Land use, topography, hydrology and political climates vary dramatically within and among the system's three watersheds: the Delaware, the Catskill and the Croton (Figure 1). One reservoir, the Kensico, is integral to managing the unfiltered Catskill and Delaware systems because it serves as the final impoundment for Catskill and Delaware water before it enters the distribution system. On average, approximately 1.3 billion gallons flow through the Kensico Reservoir each day. This accounts for 90% of the system's daily demand. For this reason, it is important to control the quality of stormwater entering the reservoir from developed land.

The U.S. Environmental Protection Agency recognizes the importance of the Kensico Reservoir and has required the NYCDEP to implement a watershed management and protection plan that targets fecal coliform bacteria and turbidity. Plan elements include aggressive stormwater and waterfowl management programs, sewer system inspection and repair, an in-reservoir turbidity curtain, and hazardous spill containment. This paper focuses solely on the stormwater management element which includes hazardous spill containment. Brief descriptions of the other components follow.

Sewer Inspection and Repair

The sewer system within the watershed, including type and size of pipe and manhole locations, was mapped. Of the 95,000 feet of sewer line in the watershed, 55,000 feet were installed before 1970 and are more prone to defects. The older sections of sewer line were inspected for potential sources of exfiltration, and cross or illicit connections. No illicit connections were discovered; 39 segments and three manholes were found to be in need of repair. The town of Mount Pleasant and Westchester County are completing the repairs under intermunicipal agreements.

Waterfowl Management

The waterfowl management program is designed to eliminate or reduce the numbers of geese and gulls roosting and defecating in or near the surface water through hazing, using noisemakers, motorboats, hovercraft, and bird distress tapes; shoreline meadow management and physical barriers; and Canada geese egg depredation. The program, implemented August 1 through March 31, also in-

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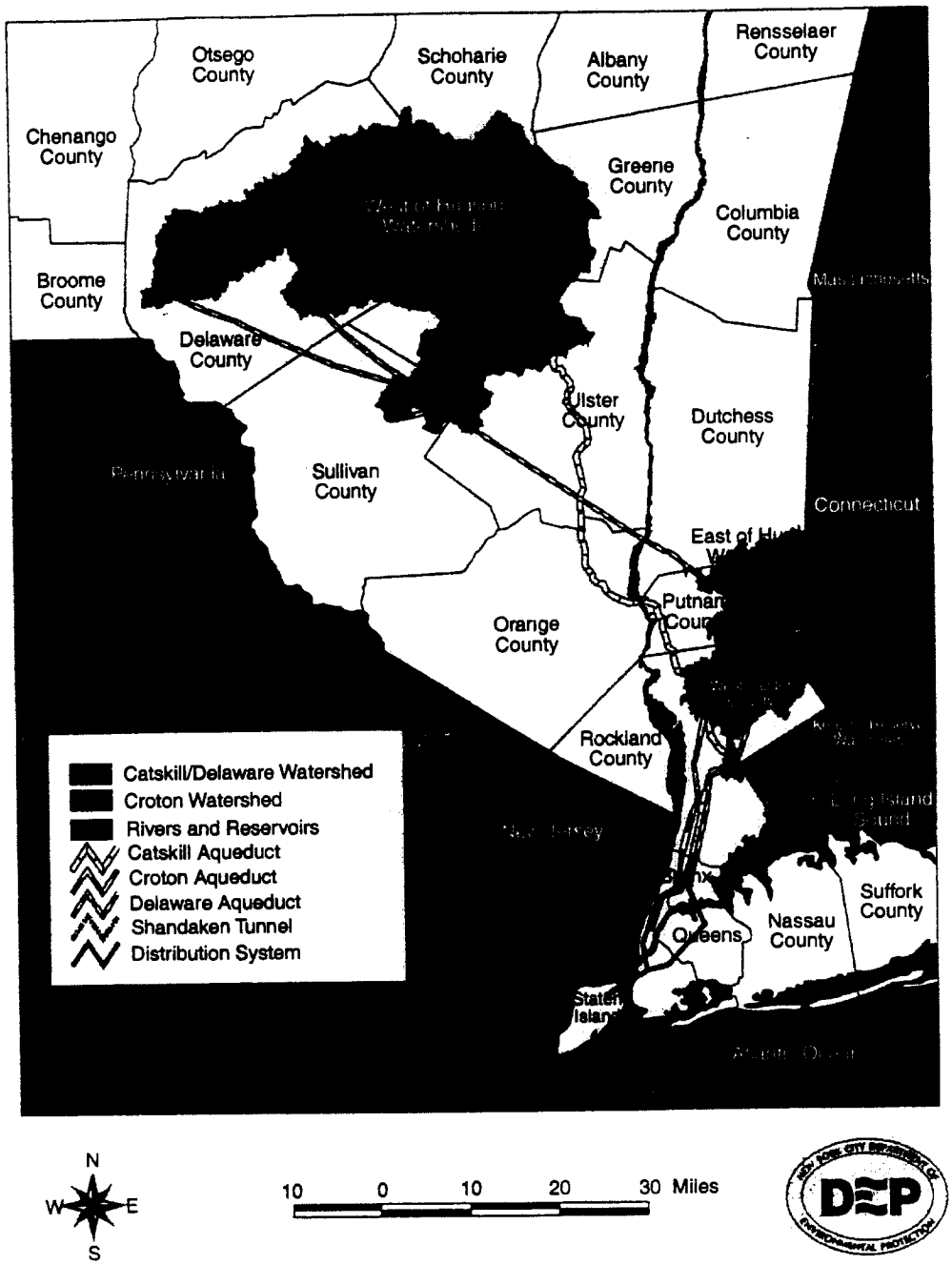

Figure 1. New York City's water supply system.

cludes research into new methods of bird control and ongoing assessments of program effectiveness. Although labor intensive, the waterfowl management program is a permanent program because it eliminates the greatest source of fecal coliform bacteria.

Turbidity Curtain

The turbidity curtain (like a silt fence 750 feet long with floats on top and weights on the bottom) installed at the mouth of Malcolm Brook in the southwest section of the Kensico Reservoir successfully directs turbidity and fecal coliform bacteria conveyed to the reservoir away from the Catskill Upper Effluent Chamber. Maintaining high-quality water in the effluent chamber is critical as water is conveyed directly to the distribution system from the chamber. Water entering the chamber is constantly monitored to determine compliance with the SWTR. Since the curtain has been so effective, it is a permanent program.

Reservoir Dredging

The channels leading to the reservoir's two effluent chambers, and the sediment deltas at the mouths of Malcolm

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and Young Brooks will be dredged in 1998. Dredging will eliminate the potential for accumulated sediments to be resuspended during storms, potentially causing a contravention of turbidity and fecal coliform bacteria water quality standards.

The Kensico Reservoir Stormwater Management Project is designed to reduce fecal coliform bacteria and turbidity delivered to the reservoir by controlling and treating stormwater. As turbidity is a direct measure of suspended solids in the water column, the stormwater component targets sources of suspended solids. The first phases of the project, assessment of the watershed, site selection, and the screening and design of stormwater control and treatment facilities, were completed in December 1997. Funding is in place to construct the stormwater facilities over a 5-year period, beginning in March 1998. The NYCDEP has committed to maintaining, monitoring, and evaluating the performance of the facilities.

Phase I: Watershed Assessment

The Kensico Reservoir watershed occupies approximately 13 square miles and includes four suburban towns in Westchester County, New York, plus a small portion in Fairfield County, Connecticut (Figure 2). To assess stormwater pollutant loading in the Kensico watershed, the reservoir basin's physical characteristics, including land use, soils, topography, vegetation and reservoir tributaries, were inventoried and digitally mapped. The watershed's topography is hilly and rolling, and over two-thirds of it contains slopes greater than 8%. Almost one-third of the land area is used as passive open space, and approximately one-fifth of the land area is developed with low-density residential uses (Figure 2). The remaining land area is primarily active open space, farmland and commercial/business. As water quality is, in part, a function of the amount of impervious surfaces in the watershed, of greatest concern is developed land directly adjacent to the effluent chambers that convey drinking water to the consumers.

Phase II: Stormwater Remediation Needs Assessment and Management Plan Development

A preliminary assessment of stormwater remediation needs in the Kensico watershed was conducted by evaluating tributary water quality data, land use/impervious surfaces, SWMM model predictions of runoff quantity and quality, and field observations of existing erosion. That evaluation concluded that 73 of the watershed's 148 sub-basins have a relatively high potential to contribute fecal coliform bacteria and suspended solids to the reservoir. Using the criteria listed below, reservoir tributaries in 19 of the 73 sub-basins were prioritized for stormwater remediation.

Preliminary Stormwater Remediation Evaluation Criteria

- proximity to reservoir effluent chambers
- known or potential sources of pollutants
- quality and quantity of stormwater runoff
- presence of wetlands
- topography
- property ownership
- observed erosion

Based upon these criteria, conceptual designs were prepared for 88 stormwater management facilities and erosion controls within 19 sub-basins. The conceptual designs were the basis for an environmental evaluation and impact statement.

The conceptual stormwater management plan was then refined by applying the selection criteria (bulleted below) in combination with the results of detailed field investigations, maintenance requirements and site constraints. A total of 57 stormwater management facilities were sited to reduce erosion, manage peak stormwater flows, allow for settling of sediments and coliform die-off, and ultimately reduce pollutant loads delivered to the reservoir (Figure 3). During the process of developing preliminary facility designs, property owners required that five facilities be redesigned, and denied permission to construct the facilities at three sites. Ultimately, 44 engineered designs were finalized. Facility types included 10 extended detention basins, 14 segments of stream channel stabilization, 13 stabilized outlets, one area of parking lot stabilization, and one sand filter system. Road stabilization and drainage improvements were incorporated into stilling and detention basins and sand filter designs. Hazardous spill containment is being addressed in coordination with a major road improvement project that will significantly alter drainage along the Interstate 684 and Route 120 corridors which abut the reservoir. The conceptual plan includes four extended detention basins that will serve as spill containment facilities, and containment booms to be deployed at the 22 storm drain outlets along I-684 in the event of a spill.

Site Selection and Conceptual Facilities Evaluation Criteria

- Do the site and the facility meet the intent of reducing pollutant loads?
- Does the facility minimize impact to environmental resources and achieve measurable water quality benefits?
- Does the existing condition warrant engineered improvements?
- Are there property ownership/permission constraints which make implementation impractical or impossible?
- Have any watershed/land use conditions or assumptions changed since issuance of the Final Environmental Impact Statement which affect the appropriateness of the facility and/or the site?
- Are there likely to be permit issues which will compromise the viability of the practice?

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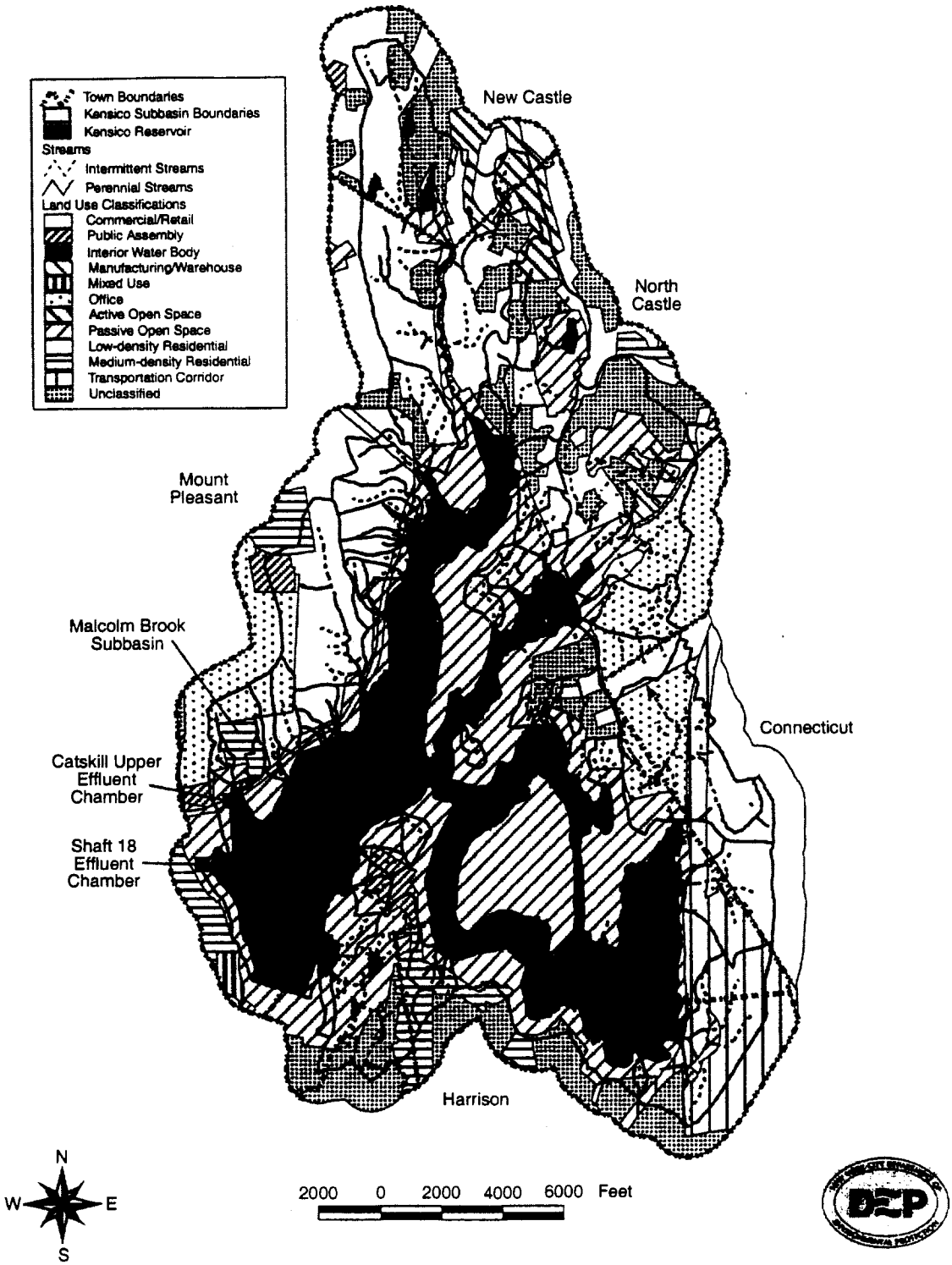

Figure 2. Land use in the Kensico Reservoir Watershed.

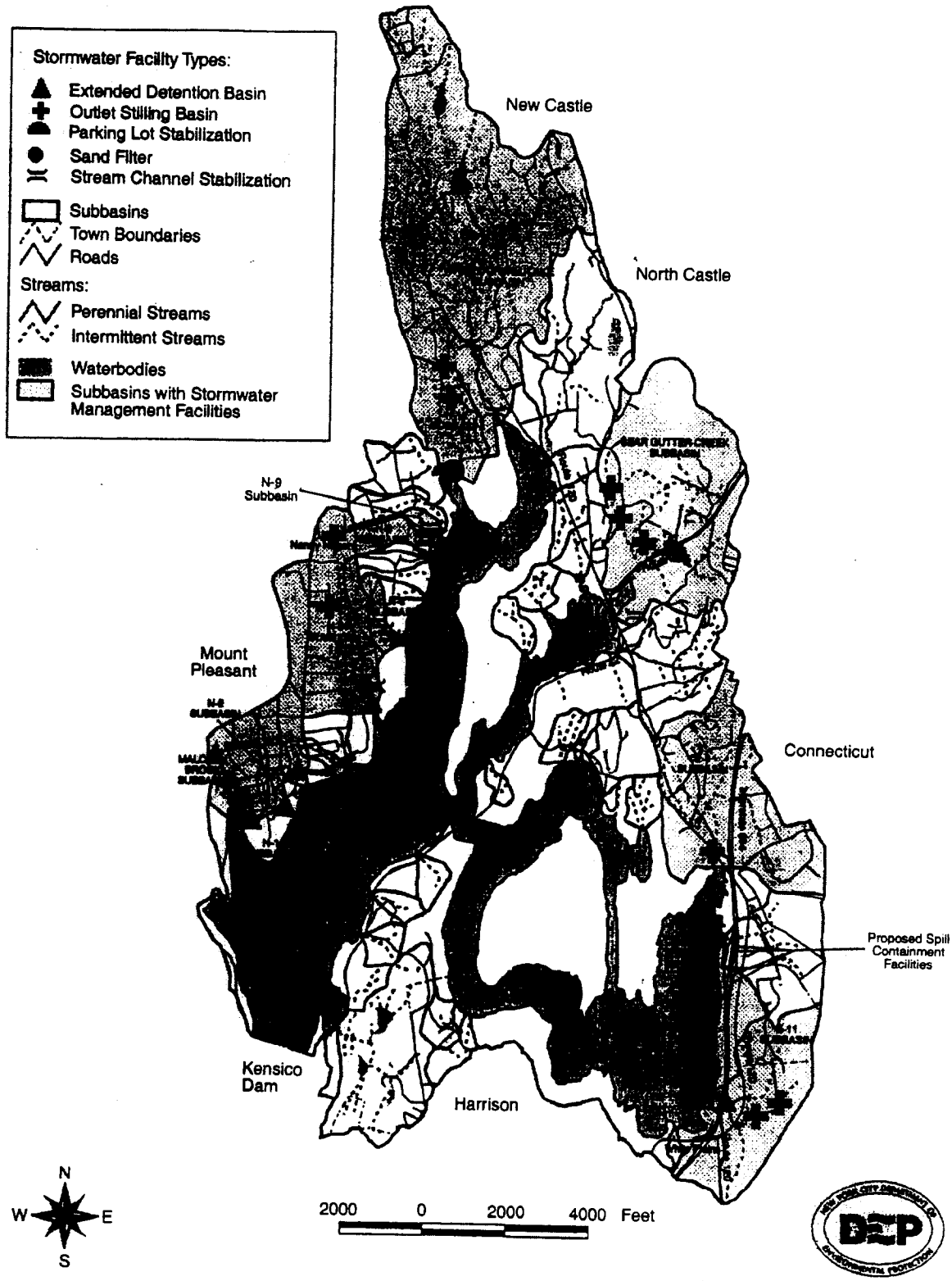

Figure 3. Proposed stormwater management facility sites in the Kensico Reservoir watershed.

- Are the maintenance and/or operation requirements of the practice, so burdensome as to effectively make the practice inappropriate?

Having met the final screening criteria, each facility was designed to minimize on-site and downstream impacts without sacrificing water quality benefits. For example, the designs incorporated existing topography, avoided wetland encroachment, and incorporated emergent wetland features necessary for long-term maintenance and features to discourage waterfowl attraction. The attention given to minimizing disturbances and subsequent on- and off-site impacts was a crucial component of enlisting the support of the community, regulatory agencies and private property owners.

Phase III: Implementing the Stormwater Management Plan in a Developed Area

Generating Community Support

The final plan proposes to construct 44 stormwater management facilities and erosion controls on public and private property. Immediately after proposing the 88 conceptual facilities, NYCDEP identified the property owners, and launched an outreach campaign to explain and generate support for the project. The ultimate goal of the campaign was to secure legal permission to gain access to design, construct, and maintain the facilities on private property. Securing permission to construct 18 facilities on private land holdings from 32 land owners has been a challenging aspect of the project. Alternate sites in the same sub-basin were pursued where access to private property was denied during preliminary design development.

Obtaining Regulatory Approvals

An expert advisory panel was enlisted to review conceptual plans and facility designs for the highest priority sub-basin, Malcolm Brook, which discharges in the immediate proximity of the Catskill Upper Effluent Chamber. Panel members were volunteers from academia and government agencies that are actively involved in planning and implementing stormwater management projects. The panel's comments helped shape the plans and designs, and were used to support certain aspects of the plan when applying for regulatory approvals.

Municipal support for the project and regulatory approval to construct the facilities were also needed. Initially, this involved a series of explanatory meetings with the town supervisors, engineers and planners. Once support for the conceptual project was obtained, the standard applications for local permits and approvals were submitted. A similar process of "pre-application" meetings was followed with federal and state permitting agencies. The pre-application meetings set the stage for the relationship between NYCDEP, the municipalities and regulatory agencies, and allowed the agencies to comment on the designs before they were finalized and permit applications were submitted. The goal of the pre-application process was to minimize the need for design revisions and to avoid delays during the regulatory approval process.

Modeling Water Quality Benefits

Water quality modeling predictions can provide valuable supporting information when developing stormwater management plans, if sufficient data are available. The U.S. Environmental Protection Agency's Stormwater Management Model (SWMM) was used to simulate runoff volumes and turbidity and fecal coliform bacteria loading in select tributary sub-basins of the Kensico Reservoir. The model predicted pollutant loads under existing conditions and future build-out conditions in the year 2010, with and without the project. With the projected increase in impervious surfaces, results from modeling estimated that future stormwater loads of turbidity and fecal coliform bacteria inputs to the reservoir will increase by 16% and 21%, respectively. Model predictions also estimated that construction and operation of stormwater facilities will reduce future inputs of turbidity and fecal coliform bacteria by 23% and 15%, respectively, when compared to future loads without the stormwater controls. The overall effect of the program on reservoir water quality will be less than the benefit associated with the targeted tributaries. Thus, for example, the model predicts much higher reductions in turbidity at the discharge of Malcolm Brook, than for the reservoir as a whole, 95% and 9.9%, respectively. Model predictions of anticipated water quality benefits in individual basins are listed in Table 1. In addition to the predicted reductions, the extended detention basins will attenuate peak rates of stormwater discharge and reduce peak concentrations of pollutants delivered to the reservoir. The results predict that the plan will have substantial water quality benefits.

Constructing, Operating, Maintaining and Monitoring Facilities

NYCDEP recognizes the need for an aggressive maintenance program to ensure that the stormwater facilities function as originally designed. Prior to construction, inspection and maintenance plans and contracts to carry out the plans will be in place. Further, water quality monitoring stations have been incorporated into facility designs and studies have been designed that will evaluate the performance of the stormwater controls.

The stormwater plan will be implemented sub-basin by sub-basin, with construction scheduled to begin in the spring of 1998. The construction schedule was prioritized using criteria that included severity of erosion, water quality benefits, proximity to the effluent chambers, and permitting and property owner constraints.

Conclusions, Recommendations and Challenges

The Kensico Reservoir watershed stormwater management plan will improve water quality in the reservoir by controlling and treating stormwater runoff in select tributaries. An aggressive public outreach campaign, designing the facilities to minimize site and resource disturbances, and providing for proper maintenance of stormwater controls and monitoring effectiveness, were high priorities for

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Table 1. SWMM Model Predictions of Tributary Loads of Turbidity and Fecal Coliform Bacteria

Tributary Sub-basin	Future Load without Plan		Future Load with the Plan	
	Turbidity	Fecal Coliform Bacteria	Turbidity	Fecal Coliform Bacteria
Malcolm Brook	5% increase	6% increase	95% reduction	72% reduction
N1	104% increase	95% increase	91% reduction	60% reduction
N2	9% increase	11% increase	81% reduction	41% reduction
N3	14% increase	10% increase	63% reduction	38% reduction
N4	103% increase	106% increase	90% reduction	52% reduction
N5	30% increase	23% increase	84% reduction	54% reduction
N12	107% increase	not modeled	68% reduction	not applicable
Bear Gutter Creek 5	60% increase	59% increase	77% reduction	59% reduction
Bear Gutter Creek 8	76% increase	73% increase	95% reduction	64% reduction
Whippoorwill	11% increase	not modeled	11% increase	not applicable
E11	0% increase	0% change	96% reduction	70% reduction

NYCDEP. The stormwater management plan will be used as a template for similar efforts in other urban reservoir watersheds, and in NYCDEP's overall stormwater management, mitigation, and cost-sharing programs. Program recommendations are as follows:

- An aggressive outreach campaign is needed to secure support for the project, get permission to include privately-owned land in the retrofit program, and obtain regulatory permits and approvals. The campaign should begin during conceptual plan development and continue through facility construction and operation.
- The pre-application review process can streamline the permitting and approval process.
- Water quality modeling results can support the selection and prioritization of sites and facility types.
- The inspection and maintenance requirements should be defined and commitments to carry out the requirements should be obtained prior to construction.
- The advisory panel formed to review conceptual plans and facility designs should be fully informed of watershed conditions, jurisdictional constraints and agency capabilities.
- Design and construction contract bid documents and payment processes should be clearly defined such that all parties itemize work units in the same manner.
- A contractor should be selected that is experienced in watershed assessment, as well as application of the remediation programs likely to be warranted in the area.
- Facility designs should maximize water quality benefits and minimize disturbances to natural resources. Water quality monitoring capabilities should be included in facility designs.

Acknowledgments

Roy F. Weston, Inc. of New York (Valhalla, NY) was contracted to develop the Kensico Water Quality Control Program and conceptual stormwater management plan.

Hazen and Sawyer, P.C. (Manhattan, NY) was contracted to reevaluate the conceptual stormwater management plan, prepare engineering designs and construction cost estimates for stormwater management and erosion control in the Kensico Reservoir Watershed, and prescribe inspection and maintenance requirements.

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Empirical Modeling Approaches for Establishing Nutrient Loading Goals for Tampa Bay

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The Tampa Bay National Estuary Program (TBNEP) has guided and supported the development of an empirically based approach to setting external nitrogen load targets for Tampa Bay, FL. Working closely with the scientists and resource managers of the TBNEP, the authors used the available data to elucidate relationships among loadings, water quality, and the subsurface light environments. These relationships were then applied to the development of defensible pollution load management targets for a Comprehensive Conservation and Management Plan (CCMP) for Tampa Bay.

Background

Tampa Bay is the largest estuary in the state of Florida. It extends approximately 35 miles into the west central coast of Florida (Figure 1), and is 5 to 10 miles wide along the majority of its length. Surface water flow from the 2,276-square-mile watershed is provided by the Hillsborough, Palm, Alafia, Little Manatee, and Manatee Rivers and over 40 minor tributaries. The mainstem of the bay is greatly affected by the exchange of seawater and nutrients to and from the Gulf of Mexico.

The biological systems of the estuary are characterized by both submerged and emergent vegetated habitats. The emergent vegetated habitats are dominated by a mosaic of mangrove forests and saltmarshes. Seagrass meadows are the dominant submerged vegetation of the estuary, and comprise *Thalassia*, *Syringodium*, *Halodule*, and *Ruppia*.

Due to development of its watershed, Tampa Bay experienced increases in pollutant loadings, declines in water quality, and loss of seagrass acreage between 1950 and the early 1980s. Long-term observations suggested that along with direct physical destruction for development (Janicki et al., 1994), pollutant loadings to Tampa Bay, and the associated decline in water quality, have contributed to a reduction in the extent of naturally occurring seagrass meadows (Lewis et al., 1985; Avery, 1991; Lewis et al., 1991). A major part of the poor water quality impacts on seagrasses is thought to be the attenuation of downwelling sunlight by excess concentrations of phytoplankton and suspended solids in the water column. Hence, the seagrass plants do not receive enough sunlight to remain healthy, and they perish. Recent applied research concerning the role of water quality in the loss of seagrass meadows was discussed in Morris and Tomasko (1992), Stevenson et al. (1993), and Batuik et al. (1992). Figures 2 and 3 present the extent of the loss of seagrass acreage between 1950 and 1990.

Due to the cooperative efforts of local and regional resource managers, the Tampa Bay Agency on Bay Management (ABM, 1989), private interests, and concerned citizens, pollutant loadings have been reduced and water quality has been improving in the bay since 1984. With the implementation of a number of different management actions in the early 1980s, including the implementation of advanced wastewater treatment technologies, chlorophyll levels have been declining from 1985 to present (Figure 4). Research on the recovery has suggested that a lag may exist in the recovery of seagrass meadows relative to the decline in seagrasses, and that seagrasses may continue to recover at a relatively slower rate (Johansson, 1991).

In 1991, the TBNEP was initiated as a cooperative program to continue this process of reducing pollutant loads, improving water quality, and restoring lost habitat. To provide an objective focus for pollutant load management targets, the participants in the program selected the acreage of seagrass meadows in the bay as a living resource benchmark by which progress could be measured. Recent observations by the Southwest Florida Water Management District have indicated that seagrass acreages have increased from 1990 to 1995, following the period of improved water quality. Thus, the important management questions for the TBNEP became:

1) How many acres of seagrass should be restored to return the bay to a restored state?

2) At what level should pollutant loads be managed to reach the seagrass acreage defined by question 1?

The TBNEP developed an ad hoc political consensus regarding the first question by establishing quantitative

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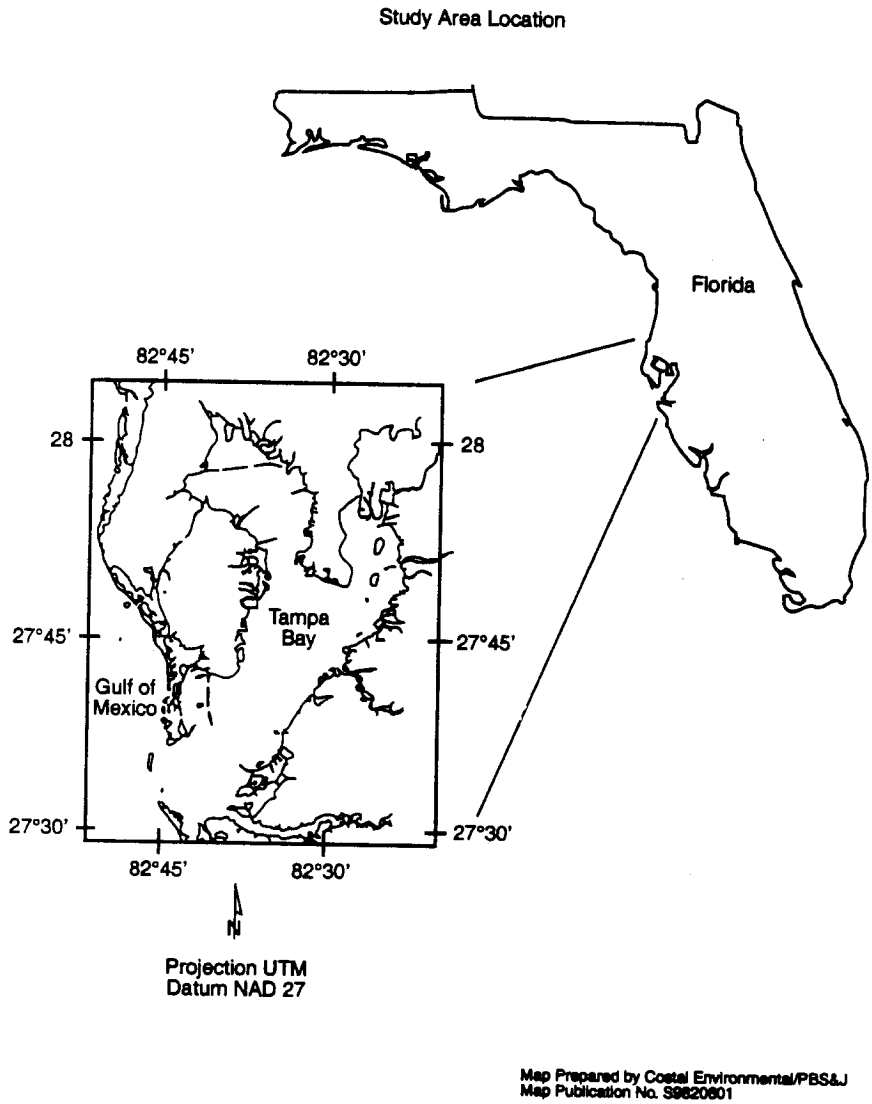

Figure 1. Tampa Bay National Estuary Program study area location.

seagrass restoration acreage targets. The target was set by mapping and enumerating the reduction in seagrass acreage from 1950 to 1990, and subtracting from this acreage the amount of seagrass that was permanently lost to physical impacts (e.g., channel dredging, island creation, borrow pits) and not likely to be restored (Janicki et al., 1994). The work for this paper was completed to answer the second question regarding what level of pollutant loads would be consistent with meeting this acreage target.

Objectives

The specific objective of this work was 1) to use the available data to document the relationships among loadings, water quality, and the subsurface light environments, and 2) to apply these empirical relationships to estimate pollution load management targets that would result in the maintenance of suitable light levels to restore the historical seagrass acreage within the bay.

A paradigm was developed to illustrate the management of nitrogen loads to effect changes in the acreage of seagrass meadows (Figure 5). Using this paradigm, external nitrogen loads to the bay result in increased nitrogen concentrations in the bay. The increased nitrogen concentrations lead to increased chlorophyll concentrations. The increased chlorophyll concentrations result in decreased depths to which surface light can penetrate in sufficient levels to maintain seagrass meadows, and the

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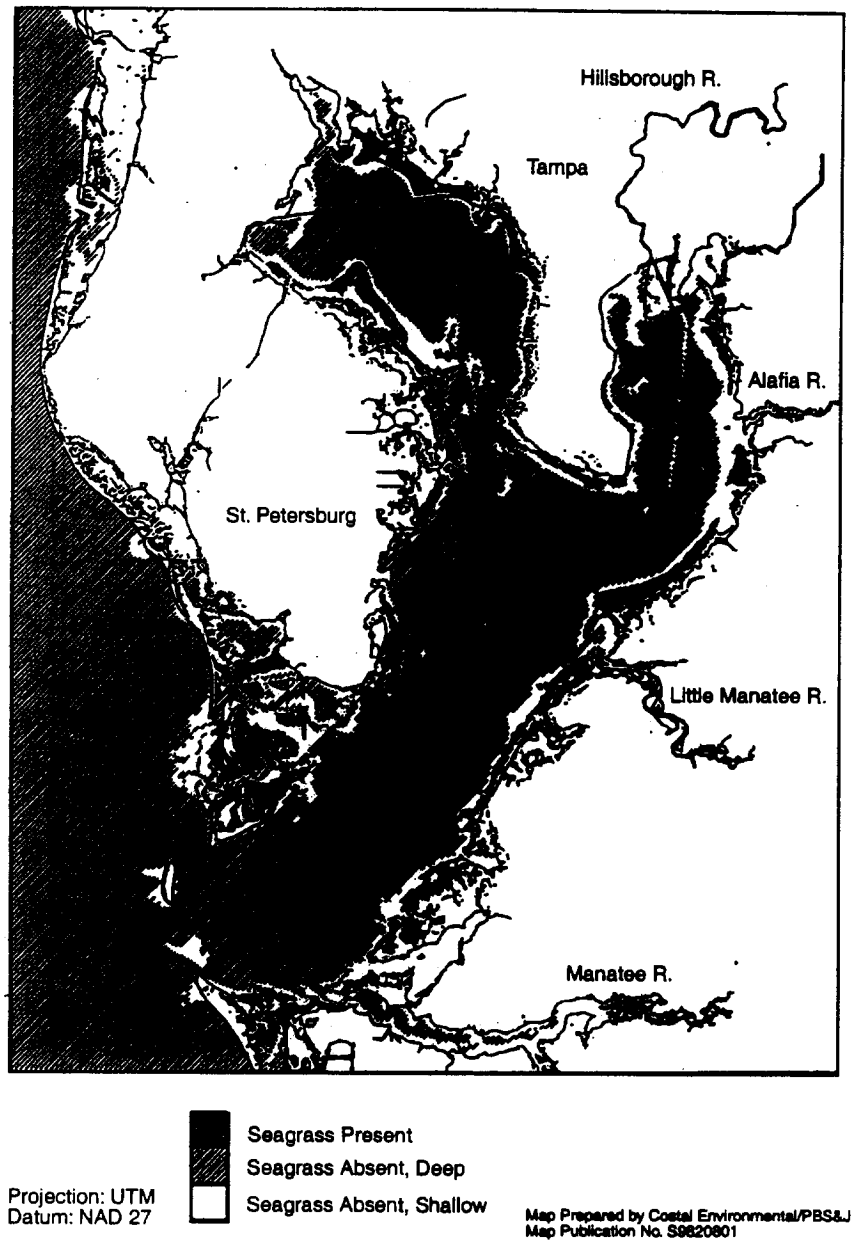

Figure 2. 1950 seagrass distribution in Tampa Bay.

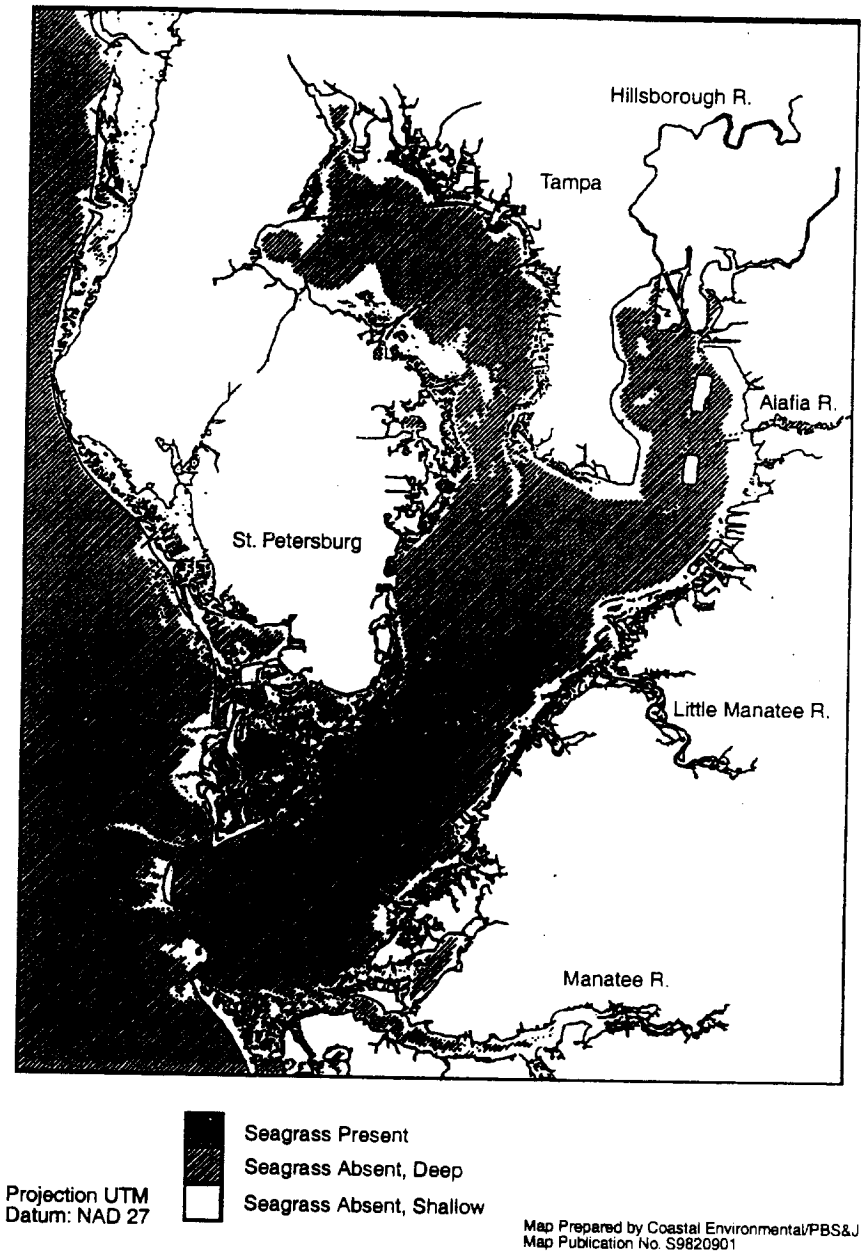

Figure 3. 1990 seagrass distribution in Tampa Bay.

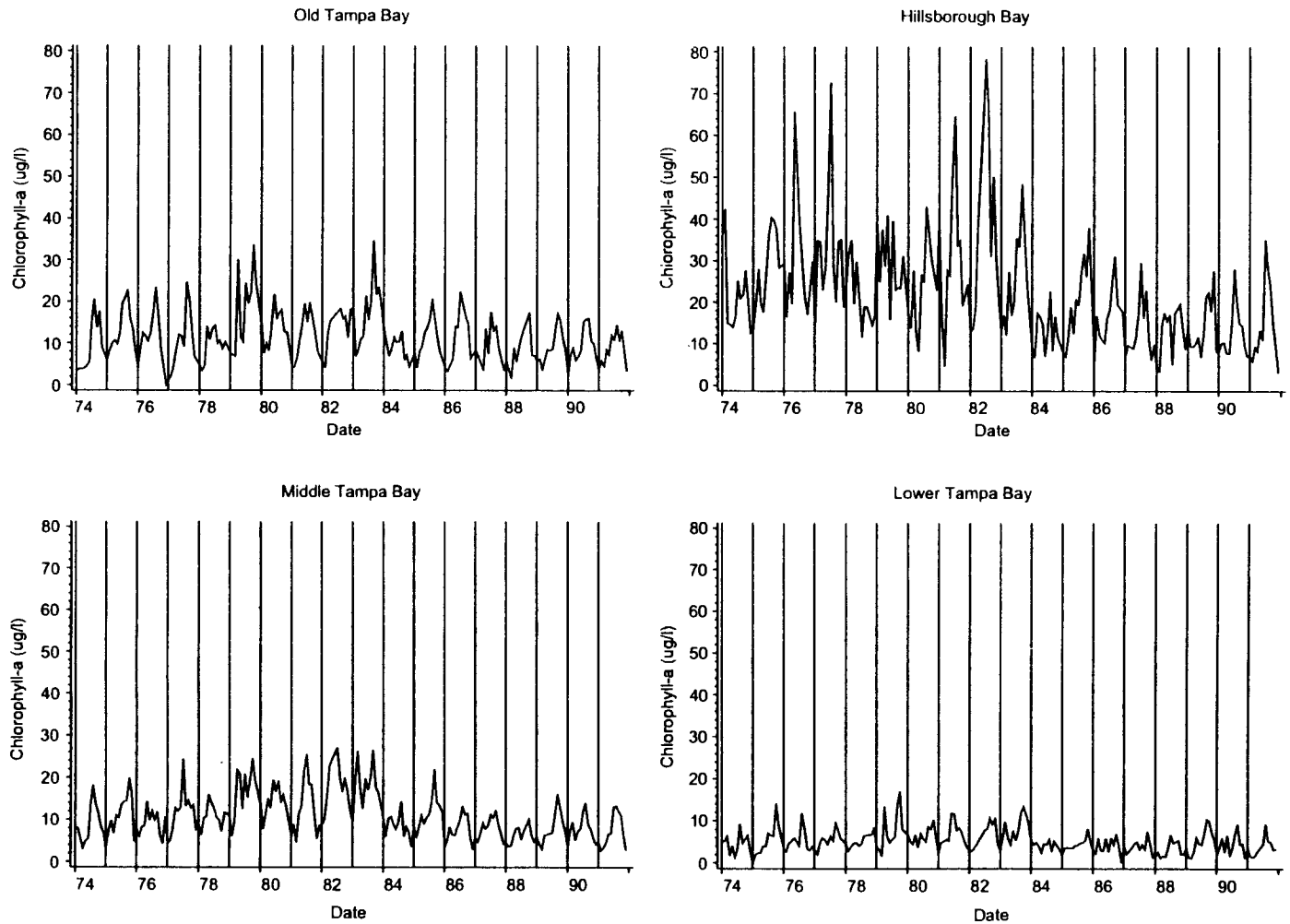

Figure 4. Historical trends in chlorophyll in Tampa Bay.

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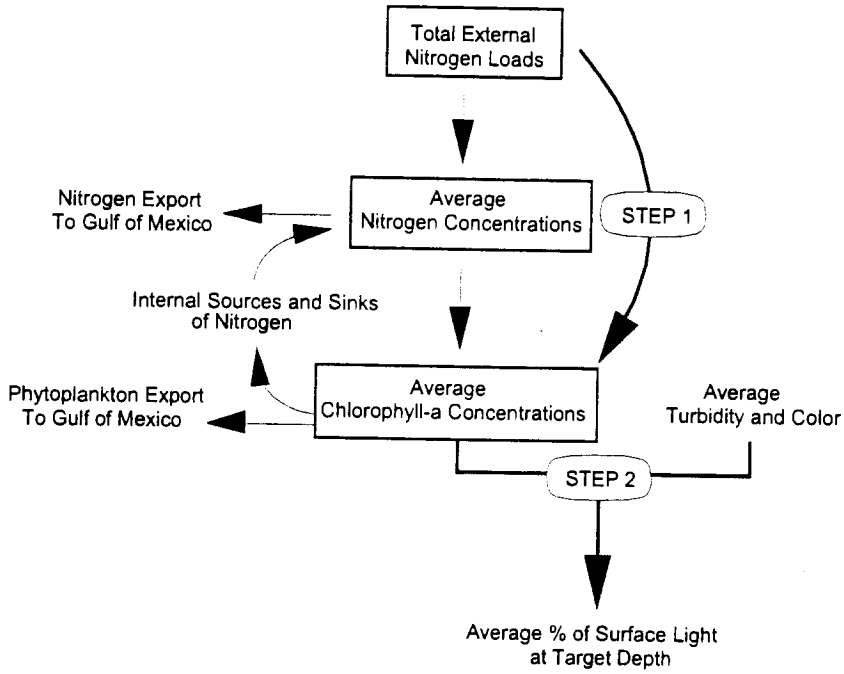

Figure 5. Empirical target setting approach paradigm.

decreased light levels result in reductions in the extent of seagrass meadows starting with the deeper meadows.

Thus, by using the available data to elucidate the links of this paradigm, the TBNEP was able to establish nitrogen load targets that would provide suitable light conditions to a desired number of acres of bay bottom that could potentially support seagrass meadows.

Methods

The first step in meeting this objective was to use the available data to quantify each of the links in the paradigm. Monthly external nitrogen load data were calculated and compiled by the TBNEP (Zarbock et al., 1994). Monthly water quality data, including nitrogen concentrations, chlorophyll concentrations, and light penetration data were used from 1986 to 1990, from data recorded by the Environmental Protection Commission of Hillsborough County. Seagrass data from the Southwest Florida Water Management District, restoration target areas from the TBNEP, and bathymetric data from the National Oceanographic and Atmospheric Administration (NOAA) were combined to quantify the restoration areas.

The final step in the approach was to apply the quantitative relationships of the paradigm to compute the nitrogen loading targets which were consistent with meeting the seagrass acreages targets.

Results

The available monthly data were sufficient to develop the links of the previously described paradigm.

Paradigm Link of Nitrogen Loads to Nitrogen Concentrations

A significant and useful relationship between monthly external nitrogen loads and monthly nitrogen concentrations in the bay (i.e., the first link in the paradigm) was not observed in the available data. Thus, a modification to the original paradigm was made to link the external nitrogen loads directly to chlorophyll concentrations. The relationships between nutrient loads and water quality have been the subject of classical limnological research (Vollenweider 1968, 1975, 1976) and more recent research for estuaries (Riley 1972, Boynton et al., 1982).

Paradigm Link of Nitrogen Loads to Chlorophyll Concentrations

Statistically significant relationships between monthly external nitrogen loads and monthly chlorophyll concentrations were observed in the available data, and were enumerated using data from 1986 to 1994. Monthly specific intercepts were used in the regression models to preclude problems with potential seasonal autocorrelation in the data. Figure 6 presents an example of these relationships. The overall R-square value for these relationships was 0.69.

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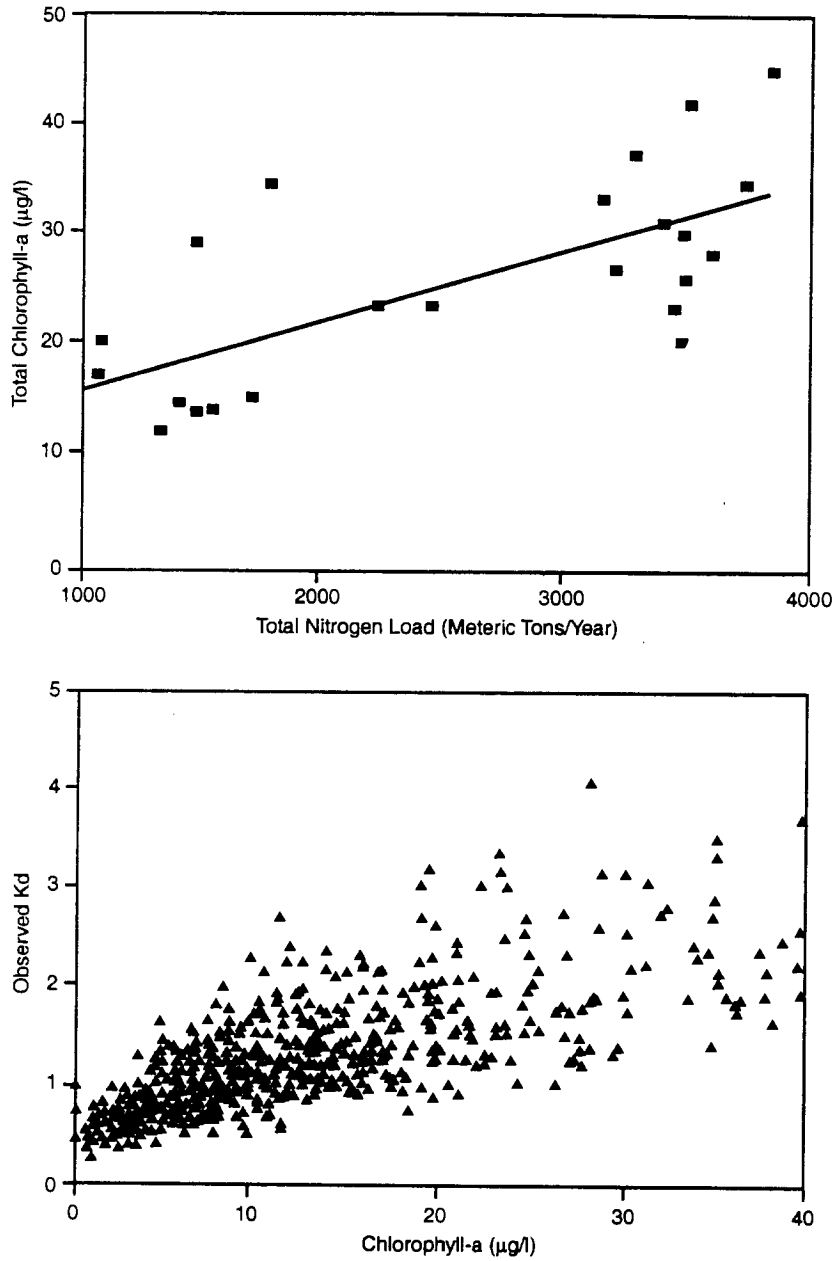

Figure 6. Resulting empirical relationships for links of the paradigm: a) the relationship of external nitrogen load to chlorophyll concentration in Hillsborough Bay, and b) the relationship of chlorophyll concentration to light attenuation in Hillsborough Bay.

The bay was divided into four segments for all steps of the paradigm. For each segment, the quantity of external nitrogen delivered to it was calculated as the input from the watershed and airshed plus the amount contributed through circulation from the other bay segments. The nitrogen loads from the watershed and airshed were calculated in a separate TBNEP study (Zarbock et al., 1994). The nitrogen contributed from circulation from the other bay segments was calculated by solving a series of steady-state dilution equations, using salt as a conservative tracer substance, and the observed salinity data for each segment and the Gulf of Mexico with the freshwater inflow data for each segment. An empirically fit parameter was used to correct these salt-derived equations for the nonconservative nature of nitrogen. This was done by comparing total nitrogen concentrations predicted by applying the conservative substance-derived dilution equations to actual nitrogen loads and comparing the predicted nitrogen concentrations with observed nitrogen concentrations. These relationships were statistically significant (overall R-square of 0.68), and indicated that for the period of 1986 to 1994, an average annual amount of approximately 8 to 9% more nitrogen was input to the upper segments of the bay than could be explained by loadings alone. A possible

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explanation for this observation is that nitrogen from the fine sediments of these segments was being liberated to the waters above. The nitrogen in the sediments likely accumulated during the pre-1985 period, during which external loads were higher.

Paradigm Link of Chlorophyll Concentrations to Light Attenuation

Statistically significant relationships were observed between observed monthly chlorophyll concentrations and light attenuation. As described below, the depth to which 20.5% of surface light is estimated to penetrate was selected as the light attenuation measurement for these relationships. As for the previous regression models, monthly specific intercepts were used to preclude potential seasonal autocorrelation in the observed data. Figure 6b presents an example of the relationship between these data. The overall R-square value for these relationships was 0.67. The TBNEP recognized that turbidity was also an important determinant of light penetration in the bay. However, chlorophyll concentrations are currently of greater concern to bay managers, and a very useful and statistically significant model could be developed using chlorophyll alone as an explanatory variable.

Paradigm Link of Light Attenuation to Seagrass Restoration

Because a large database of light attenuation data paired with seagrass condition data was not available, this final link of the paradigm was derived from observations from independent research conducted by the participating TBNEP scientists. A TBNEP study conducted for the Southwest Florida Water Management District by Mote Marine Laboratory (Dixon and Leverone, 1995) collected photosynthetically active radiation (PAR) measurements at the deep edges of seagrass beds in the lower portion of the bay for one year. After subtracting an average measured bottom reflectance of 2%, the TBNEP scientists reached a consensus that an annual average 20.5% of surface light was needed for seagrass survival in Tampa Bay.

Application of the Results to Nitrogen Load Targets

The empirically derived links of the paradigm were applied as a single tool to answer the second question defined above (i.e., at what level should nitrogen loads be managed to meet the seagrass acreage restoration targets set by the TBNEP).

A series of candidate 15-year nitrogen load schedules were applied to the integrated models and the resulting chlorophyll values were estimated. These chlorophyll values were then used to estimate the depth to which 20.5% of surface light would penetrate. The NOAA depth data and seagrass restoration areas were then applied to a detailed computer mapping system to calculate the acreage of seagrass restoration areas which would be illuminated at 20.5% of surface light under the estimated chlorophyll levels. As described previously, seagrass restoration areas were defined as those areas of the bay bottom which had seagrass meadows in 1950, did not have seagrass meadows in 1990, and were not physically altered so as to preclude the restoration of seagrass given a proper light regime.

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Alum Treatment of Stormwater Runoff - An Innovative BMP for Urban Runoff Problems

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Alum treatment of stormwater runoff was begun in 1986 as part of a lake restoration project at Lake Ella in Tallahassee, Florida in 1986. Our system provides treatment of stormwater runoff entering the lake by injecting liquid alum into major stormsewer lines on a flow-weighted basis during rain events. When added to runoff, alum forms non-toxic precipitates of $Al(OH)_3$ and $AlPO_4$ which combine with phosphorus, suspended solids and heavy metals, causing them to be rapidly removed from the treated water. The alum stormwater treatment system resulted in immediate and substantial improvements to water quality in Lake Ella which led to implementation of similar systems on other urban lakes. There are currently 23 alum stormwater treatment systems either operational or under construction in Florida, and one experimental system in Seattle, Washington.

Alum treatment of stormwater runoff has consistently achieved a 90% reduction in total phosphorus, 50-70% reduction in total nitrogen, 50-90% reduction in heavy metals, and >99% reduction in fecal coliform. Ultimate water quality improvements in the receiving water body have been related to the percentage of total inputs treated by the system. Heavy metal and phosphorus associations with alum floc have been shown to be extremely stable over a wide range of pH and redox conditions.

In general, alum treatment of runoff is substantially less expensive than traditional treatment methods and often requires no additional land purchase. Recent designs have incorporated automatic floc collection and removal systems with disposal to drying beds or sanitary sewer.

Introduction

The addition of alum to water results in the production of chemical precipitates which remove pollutants by two primary mechanisms. Removal of suspended solids, algae, phosphorus, heavy metals and bacteria occurs primarily by enmeshment and adsorption onto aluminum hydroxide precipitate according to the following net reaction:

$$Al^{+3} + 6H_2O \rightarrow Al(OH)_{3(s)} + 3H_3O^+$$

Removal of additional dissolved phosphorus occurs as a result of direct formation of $AlPO_4$ by:

$$Al^{+3} + HnPO_4^{n-3} \rightarrow AlPO_{4(s)} + Hn^+$$

The aluminum hydroxide precipitate, $Al(OH)_3$, is a gelatinous floc which attracts and adsorbs colloidal particles onto the growing floc, thus clarifying the water. Phosphorus removal or entrapment can occur by several mechanisms, depending on the solution pH. Inorganic phosphorus is also effectively removed by adsorption to the Al(OH)3 floc. Removal of particulate phosphorus is most effective in the pH range of 6-8 where maximum floc occurs (Cooke and Kennedy, 1981). At higher pH values, OH^- begins to compete with phosphate ions for aluminum ions, and aluminum hydroxide-phosphate complexes begin to form. At lower pH values and higher inorganic phosphorus concentrations, the formation of aluminum phosphate ($AlPO_4$) is favored.

In 1985, a lake restoration project was initiated at Lake Ella, a shallow 13.3-acre hypereutrophic lake in Tallahassee, Florida, which receives untreated stormwater runoff from approximately 163 acres of highly impervious urban watershed area. Initially, conventional stormwater treatment technologies, including retention basins, exfiltration trenches and filter systems, were considered for reducing available stormwater loadings to Lake Ella in an effort to improve water quality within the lake. Since there was no available land surrounding Lake Ella that could be used for construction of traditional stormwater management facilities, and the purchasing of homes and businesses to acquire land for construction of these facilities was cost-prohibitive, alternate stormwater treatment methods were considered.

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Chemical treatment of stormwater runoff was evaluated using various chemical coagulants, including aluminum sulfate, ferric salts, and polymers. Aluminum sulfate (alum) consistently provided the highest removal efficiencies and produced the most stable end product. In view of successful jar test results on runoff samples collected from the Lake Ella watershed, the design of a prototype alum injection stormwater system was completed. Construction of the Lake Ella alum stormwater treatment system was completed in January 1987, resulting in a significant improvement in water quality.

The alum precipitate formed during coagulation of stormwater can be allowed to settle in receiving waterbodies or collected in small settling basins. Alum precipitates are exceptionally stable in sediments and will not redissolve due to changes in redox potential or pH under conditions normally found in surface waterbodies. Over time, the freshly precipitated floc ages into even more stable complexes, eventually forming gibsite. The solubility of dissolved aluminum in the treated water is regulated entirely by chemical equilibrium. As long as the pH of the treated water is maintained within the range of 5.5-7.5, dissolved aluminum concentrations will be minimal. In many instances, the concentration of dissolved aluminum in the treated water will be less than the concentration in the raw untreated water due to adjustment of pH into the range of minimum solubility.

Since the Lake Ella system, alum stormwater treatment systems have been constructed in Florida for Lake Dot, Lake Rowena and Lake Lucerne in Orlando; Lake Osceola, Lake Virginia North and Lake Mizell in Winter Park; Lake Cannon in Polk County; Channel 2 Drainage Canal in Pinellas Park; Celebration Town Lake in Celebration; Lake Holden in Orange County; Lake Tuskawilla in Ocala; and a set of five separate systems has been installed at Lake Maggiore in St. Petersburg. An experimental treatment facility has also been constructed in the Lake Sammamish watershed in Seattle, Washington. In addition to these projects which are currently operational, additional projects are currently under design in Winter Park, Orlando, Largo, Tampa and Clearwater. The first project to treat stormwater discharged to a brackish water became operational in January 1998 in the City of St. Petersburg, Florida.

Alum treatment of stormwater runoff has now been used as a viable stormwater treatment alternative in urban areas for over 10 years. Over that time, a large amount of information has been collected relative to optimum system configuration, water chemistry, sediment accumulation and stability, construction and operation costs, comparisons with other stormwater management techniques, and floc collection and disposal. A summary of current knowledge in these areas is given in the following sections.

System Configuration

Once alum has been chosen as an option in a stormwater retrofit project, extensive laboratory testing must be performed to verify feasibility and to establish design parameters. The feasibility of alum treatment for a particular stormwater stream is typically evaluated in a series of laboratory jar tests conducted on representative runoff samples collected from the project watershed area. This extensive laboratory testing is necessary to determine design, maintenance and operational parameters such as the optimum coagulant dose required to achieve the desired water quality goals, chemical pumping rates and pump sizes, the need for additional chemicals to buffer the pH of receiving water, post-treatment water quality characteristics, floc formation and settling characteristics, floc accumulation, annual chemical costs and storage requirements, ecological effects, and maintenance procedures. In addition to determining the optimum coagulant dose, jar tests can also be used to determine floc strength and stability, required mixing intensity and duration, and design criteria for dedicated floc settling basins.

In a typical alum stormwater treatment system, alum is added to the stormwater flow on a flow-proportional basis so that the same dose of alum is added to a gallon of stormwater flow regardless of the discharge rate. A variable speed chemical metering pump is typically used as the injection pump. If a buffering agent, such as NaOH, is required to maintain desired pH levels, a separate metering system and storage tank will be necessary. The operation of each injection pump is regulated by a flow meter device attached to each incoming stormwater line to be treated. Data from each stormwater flow meter is transformed into a 4-20 mA electronic signal which instructs each metering pump to inject alum according to the measured flow through each individual line. Mixing of the alum and stormwater occurs as a result of turbulence in the stormsewer line. If sufficient turbulence is not available within the stormsewer line, artificial turbulence can be generated using aeration or physical stormsewer modifications.

Mechanical components for the alum stormwater treatment system, including chemical metering pumps, stormsewer flow meters and electronic controls, are typically housed in a central facility which can be constructed as an above-ground or below-ground structure. A 6,000 gallon tank is typically used for bulk alum storage. Alum feed lines and electrical conduits are run from the central facility to each point of flow measurement and alum addition. Alum injection points can be located as far as 3000 ft from the central pumping facility. Early designs for alum stormwater treatment systems utilized individual chemical metering pumps and stormsewer flow meters for each point of alum addition. However, in an effort to reduce overall system costs and complexity, current alum stormwater treatment systems often feed alum to multiple points using a single chemical metering pump and control valves.

Water Chemistry

In general, construction and operation of alum stormwater treatment systems has resulted in significant improvements in water quality for treated waterbodies. The degree of observed improvement in water quality is directly

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related to the percentage of annual hydraulic inputs treated by the alum stormwater treatment system. A comparison of pre- and post-modification water quality for three typical alum stormwater treatment systems is given in Table 1, including Lake Ella and Lake Dot (which provide treatment for approximately 95-96% of the annual hydraulic inputs entering these lake systems), and Lake Osceola (which provides treatment for only 9% of the annual hydraulic inputs entering the lake system).

Operation of the alum stormwater treatment systems resulted in a decline in pH within each of the three waterbodies, ranging from a reduction of approximately 1 unit in Lake Ella to 0.6 units in Lake Osceola. A pH reduction of only 0.1 unit was observed for the Lake Dot treatment system which injects alum in combination with sodium hydroxide to control pH levels within the lake. In addition, significant improvements in dissolved oxygen were observed in both Lake Ella and Lake Dot. Alum treatment of stormwater runoff resulted in a 78% reduction in total nitrogen concentrations in Lake Ella, a 55% reduction in Lake Dot and a 4% reduction in Lake Osceola where only a small portion of the annual hydraulic inputs are treated. The majority of the total nitrogen removal observed is a result of reducing concentrations of dissolved organic nitrogen and particulate nitrogen, since alum is generally ineffective in reducing concentrations of inorganic nitrogen species, such as ammonia or NO_x. Alum stormwater treatment resulted in a substantial reduction in measured concentrations of orthophosphorus and total phosphorus in each of the three lake systems, with total removals of 89%, 93% and 30% for Lake Ella, Lake Dot and Lake Osceola, respectively. Alum stormwater treatment also reduced in-lake concentrations of BOD in each of the three lake systems, with a reduction of 93% in Lake Ella, 84% in Lake Dot and 22% in Lake Osceola.

Alum stormwater treatment has been extremely effective in reducing concentrations of chlorophyll-a in receiving waterbodies, with a reduction of 97% in Lake Ella, 89% in Lake Dot and 13% in Lake Osceola. Reductions in measured concentrations of chlorophyll-a occur as a result of enmeshment and precipitation of algal particles within the water column of the lake by alum floc as well as phosphorus limitation created by low levels of available phosphorus in the water column. Substantial increases in Secchi disk depth were observed in Lake Ella and Lake Dot, and to a lesser extent in Lake Osceola, with improvements of 340% in Lake Ella, 212% in Lake Dot and 9% in Lake Osceola. Based upon the Florida TSI Index (Brezonik, 1984), Lake Ella and Lake Dot have been converted from hypereutrophic to oligotrophic status, with a conversion from eutrophic to mesotrophic the case in Lake Osceola.

A graphic history of total phosphorus concentrations in Lake Lucerne, which was retrofitted with an alum stormwater treatment system in June 1993 that provides treatment for approximately 82% of the annual runoff inputs into the lake, is given in Figure 1. Prior to construction of the alum stormwater treatment system, total phosphorus concentrations in Lake Lucerne fluctuated widely, with a mean concentration of approximately 100 µg/l. Following start-up of the alum treatment system, total phosphorus concentrations began to decline steadily, reaching equilibrium concentrations of 20-40 µg/l. A slight increase in total phosphorus concentrations was observed during the last half of 1995 when the system was off-line due to lightning damage. When system operation resumed in June

Table 1. Comparison of Pre- and Post-Modification Water Quality Characteristics for Typical Alum Stormwater Treatment Systems

PARAMETER	UNITS	LAKE ELLA BEFORE (1974-85)	LAKE ELLA AFTER (1/88-5/90)	LAKE DOT BEFORE (1986-88)	LAKE DOT AFTER (3/89-8/91)	LAKE OSCEOLA BEFORE (6/91-6/92)	LAKE OSCEOLA AFTER (2/93-12/96)
# of Samples	–	15	11	5	15	12	46
pH	s.u.	7.41	6.43	7.27	7.17	8.22	7.63
Diss. O_2 (1 m)	mg/l	3.5	7.4	6.6	8.8	8.8	8.8
Total N	µg/l	1876	417	1545	696	892	856
Total P	µg/l	232	26	351	24	37	26
BOD	mg/l	41	3.0	16.8	2.7	4.4	3.4
Chlorophyll-a	mg/m³	180	5.1	55.8	6.3	24.8	21.7
Secchi Disk Depth	m	0.5	> 2.2	< 0.8	2.5	1.1	1.2
Diss. Al	µg/l	–	44	–	65	18	51
Florida TSI Value	–	98 (Hyper-eutrophic)	47 (Oligotrophic)	86 (Hyper-eutrophic)	42 (Oligotrophic)	61 (Eutrophic)	56 (Meso-trophic)
Lake Area	–	13.3 ac		5.9 ac		55.4 ac	
Watershed Area	–	57 ac		305 ac		153 ac	
Percent of Annual Hydraulic Inputs Treated	%	95		96		9	

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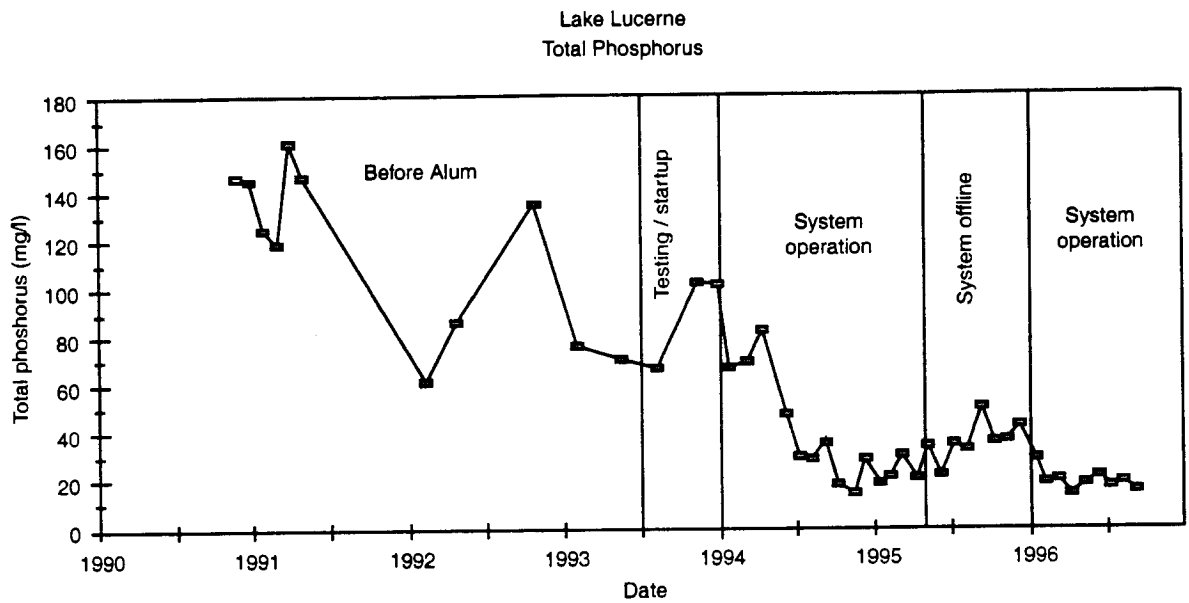

Figure 1. Trends in total phosphorus concentrations in Lake Lucerne before and after alum treatment of stormwater runoff.

1996, total phosphorus concentrations returned to equilibrium values of approximately 20 µg/l.

In general, measured concentrations of heavy metals have been extremely low in value in all waterbodies retrofitted with alum stormwater treatment systems, with no violations of heavy metal standards. In addition, measured levels of dissolved aluminum have also remained low in each lake system. Mean dissolved aluminum concentrations for Lake Ella, Lake Dot and Lake Osceola have averaged 44 µg/l, 65 µg/l and 51 µg/l, respectively. Although there is no standard for dissolved aluminum in the State of Florida, the U.S. EPA has recommended a long-term average of 87 µg/l for protection of all species present in the U.S. The solubility of dissolved aluminum is regulated almost exclusively by pH. As long as the pH of the treated water can be maintained in the range of 6.0-7.5 during the treatment process, dissolved aluminum concentrations will remain at minimal levels.

Floc Accumulation

Laboratory investigations have been conducted on stormwater runoff collected from a wide range of land uses typical of urban areas to quantify the amount of alum floc generated as a result of alum treatment of stormwater runoff at various treatment doses. After initial formation, alum floc appears to consolidate rapidly for a period of approximately 6-8 days, reaching approximately 20% of the initial floc volume. Additional consolidation appears to occur over a settling period of approximately 30 days, after which collected sludge volumes appear to approach maximum consolidation (Harper, 1990).

Estimates of maximum anticipated sludge production, based upon literally hundreds of laboratory tests involving coagulation of stormwater runoff with alum at various doses, and based upon a consolidation period of approximately 30 days, are given in Table 2. At alum doses typically used for treatment of stormwater runoff, ranging from 5-10 mg/l as Al, sludge production is equivalent to approximately 0.16-0.28% of the treated runoff flow. Sludge production values listed in Table 2 reflect the combined mass generated by alum floc as well as solids originating from the stormwater sample.

Field investigations have also been performed in lake systems receiving alum treated stormwater runoff to document the accumulation rate of alum floc within the sediments by visual inspection of sediment core samples collected in clear acrylic tubes at selected monitoring sites in each lake. A comparison of observed and predicted floc accumulation rates in lake systems receiving stormwater treatment is given in Table 3. Each of the listed lakes has been receiving alum treatment for five years or more. The primary predicted settling area for floc accumulation was determined by evaluating lake bottom topography and stormsewer inflow characteristics. Predicted floc accumu-

Table 2. Anticipated Production of Alum Sludge from Alum Treatment of Stormwater at Various Doses

	Sludge Production[1]	
Alum Dose (mg/l as Al)	As Percent Of Treated Flow	Per 10^6 Gallons Treated
5	0.16	214 ft^3
7.5	0.20	268 ft^3
10	0.28	374 ft^3

[1]Based on a minimum settling time of 30 days

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Table 3. Comparison of Observed and Predicted Floc Accumulation Rates in Lake Systems with Alum Stormwater Treatment

Lake	Predicted Settling Area	Predicted Accumulation Rate	Observed Accumulation Rate
Lake Ella	50% of lake bottom	1 cm/yr	0.33 cm/yr
Lake Lucerne	areas 10 ft or deeper	3.3 cm/yr	none
Lake Osceola	50% of lake bottom	0.5 cm/yr	none

lation rates are based upon the anticipated floc production rates summarized in Table 2.

Annual floc production in Lake Ella was predicted to be approximately 1 cm/yr over 50% of the lake bottom. However, floc accumulation evaluations performed in 1990 indicate an observed accumulation rate of approximately 0.33 cm/yr, approximately one-third of the predicted accumulation rate. The reduced observed accumulation rate is thought to be a result of additional floc consolidation over time and incorporation of the alum floc into the existing sediments. The observed post-treatment floc accumulation rate in Lake Ella is similar to the pre-treatment sediment accumulation rate in Lake Ella resulting from the extremely high algal production prior to the lake restoration efforts in 1985. Sediment accumulation in Lake Lucerne was anticipated to occur in areas 10 ft or deeper, with a predicted accumulation of 3.3 cm/yr. However, no sediment accumulation was observed at any of the 10 fixed monitoring locations within the lake which have contributed data on an annual basis since start-up of the alum treatment system. A similar conclusion has been reached in Lake Osceola which has no visible floc accumulation after approximately five years of alum stormwater treatment. Both Lake Lucerne and Lake Osceola appear to incorporate alum floc into the existing sediments with no visible surface floc layer.

Construction and O&M Costs

A summary of construction and annual operation and maintenance (O&M) costs for existing alum stormwater treatment facilities, with treated watershed areas ranging from 64 ac to 1450 ac, is given in Table 4. Construction costs for alum stormwater treatment systems have ranged from $75,000 to $400,000, depending upon the number of outfalls to be retrofitted. In general, the capital cost of constructing alum stormwater treatment systems is independent of the watershed size since the capital cost for constructing a treatment system for a 100-acre watershed is identical to the cost of constructing a system to treat 1000 acres at the same location, although annual O&M costs would differ. The average capital cost for existing alum stormwater treatment facilities is $245,998.

Estimated O&M costs are also provided in Table 4 and include chemical, power, manpower for routine inspections, and equipment renewal and replacement costs. Operation and maintenance costs for existing alum stormwater treatment systems range from $5,500 to $26,298 per year. Construction costs and annual O&M costs are also included on a per acre treated basis for comparison with other stormwater treatment alternatives.

Comparison with Other Stormwater Treatment Alternatives

In general, removal efficiencies obtained with alum stormwater treatment are similar to removal efficiencies obtained using a dry retention or wet detention stormwater management facility. A comparison of treatment efficiencies for common stormwater management systems is given in Table 5 (Harper, 1995). Estimated removal efficiencies for alum treatment exceed removal efficiencies achieved in dry retention for total phosphorus and TSS, but fall short of dry retention for total nitrogen and BOD. Dry retention

Table 4. Summary of Construction and O&M Costs for Existing Alum Stormwater Treatment Facilities

Project	Area Treated (ac)	Construction Cost/System ($)	Estimated Annual O&M Cost ($)	Construction Cost Per Area Treated ($/ac)	Annual O&M Cost Per Area Treated ($/ac)
Lake Ella	158	200,400	–	1268	–
Lake Dot	305	250,000	–	823	–
Lake Lucerne	272	400,000	16,000	1472	59
Lake Osceola	153	300,000	6500	1959	43
Lake Cannon	490	135,000,	13,100,	276	27
Channe 2	84	180,000	–	2144	–
Lake Virginia North	64	242,000	–	3769	–
Celebration	158	300,000	25,000	1898	158
Lake Holden	183	292,000	–	1598	–
Lake Tuskawilla	311	242,000	19,627	777	63
Lake Rowena	538	75,000	–	139	–
Lake Mizell	74	300,000	15,389	4049	208
Lake Maggiore (5)	1450	400,000	21,450	1379	74
Webster Avenue	91	130,000	12,397	1423	136
Lake Virginia South	437	288,000	–	659	–
Merritt Ridge	195	201,575	26,298	1033	135
AVERAGES	310	$ 245,998	$17,307	$1542	$100

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Table 5. Estimated Removal Efficiencies for Common Stormwater Management Systems

Type of System	Estimated Removal Efficiencies (%) Total N	Total P	TSS	BOD
Dry Retention (0.50-in runoff)	80	80	80	80
Wet Detention	20-30	60-70	85	50-60
Wet Detention with Filtration	0	60	>90	90
Dry Detention	10-20	20-40	60-80	03-50
Dry Detention with Filtration	0-20	0-20	60-90	0-55
Alum Treatment	50-70	>90	>95	60

may be the more effective common stormwater management technique in use today, if other things are equal. But certainly, removal efficiencies achieved with alum treatment exceed removal efficiencies obtained using wet detention, wet detention with filtration, dry detention, or dry detention with filtration. Alum is the best choice where space for retention cannot be built.

Alum treatment of stormwater runoff compares favorably with other stormwater treatment alternatives with respect to both initial capital construction costs and annual O&M costs. A comparison of certain costs for alum stormwater treatment and equivalent retention facilities is given in Table 6. Initial capital construction costs and annual O&M costs for three existing alum stormwater treatment facilities are compared with the estimated cost for construction of an alternate retention facility for treatment of the first 0.5 in of runoff. Each of the alternate retention facilities would require purchase of land in heavily urbanized areas which if available, would be expensive. The cost listed for the alternate retention facilities include land costs only and not actual construction costs. Estimated annual O&M costs for retention pond maintenance, such as routine mowing, weed control and trash removal, is higher than the estimated O&M costs for the alum treatment systems which include chemicals, weekly inspections, and parts and supplies.

Table 6. Comparison of Certain Costs for Alum Stormwater Treatment and Equivalent Retention Facilities

		Alum Treatment System		Equivalent Retention Facility		
Location	Area Treated (ac)	Capital Costs ($)	Annual O&M Costs[1] ($)	Land Area Required[2] (ac)	Land Costs ($)	Annual O&M Costs[3] ($)
Lake Osceola	88	235,000	6500	3.0	1,500,000[4]	9000
Lake Lucerne	210	420,000	16,000	7.3	3700,000[4]	21,900
Lake Cannon	490	135,000	13,100	17.0	850,000[5]	51,000

[1] Includes chemical costs, weekly inspection, and $1000 for supplies and parts
[2] Based on equivalent treatment of 1 inch of runoff and a 3-ft deep pond
[3] Based on $3000/acre for O&M (Ref; FDOT)
[4] Based on a land cost of $500,000/acre
[5] Based on a land cost of $50,000/acre

Floc Collection and Disposal

Although virtually all existing alum stormwater treatment systems allow for floc settling in receiving waterbodies, and although only beneficial aspects of alum floc accumulation have been observed, current alum treatment system designs feature collection and disposal of floc. Where possible, sump areas have been constructed to provide a basin for collection and accumulation of alum floc. The accumulated floc can then be pumped out of the sump area, using either manual or automatic techniques, on a periodic basis. Several current treatment systems provide for automatic floc disposal into the sanitary sewer system at a slow controlled rate. Since alum floc is virtually inert and has a consistency similar to that of water, acceptance of alum floc on a periodic basis poses no operational problem for wastewater treatment facilities. A schematic of a settling pond designed for the Lake Virginia system is included in Figure 2.

A recent design for collection of floc discharging from a submerged pipe in a lake system is also illustrated in Figure 2. The floc containment area consists of a fabric mesh sized to allow water flow while trapping floc particles. The floc is then collected in the sump area in the bottom of the containment area and pumped on a periodic basis to the sanitary sewer system or adjacent drying bed. Drying characteristics for alum sludge are similar to a wastewater treatment plant sludge. A drying time of approximately 30 days is sufficient to dewater and dry the sludge, with a corresponding volume reduction of 80-90%. Dried alum sludge has chemical characteristics suitable for general land application or in agricultural sites, as outlined in Chapter 62 of the Florida Administration Code (Florida Department of Environmental Protection, 1996).

Conclusions

Alum treatment of stormwater runoff has emerged as a viable and cost-effective alternative for providing stormwater retrofit in urban areas. Based upon 10 years of experience with alum stormwater treatment, the following conclusions have been reached:

- In lake systems where a large percentage of the annual runoff inputs are retrofitted with an alum treatment system, alum treatment has consistently achieved a 90% reduction in total phosphorus, 50-70% reduction in total nitrogen, 50-90% reduction in heavy metals, and >99% reduction in fecal coliforms. Ultimate water quality improvements in the receiving waterbodies are highly correlated with the percentage of total inputs treated by the system.

- The observed accumulation rate of alum floc in the sediments of receiving waterbodies are to be substan-

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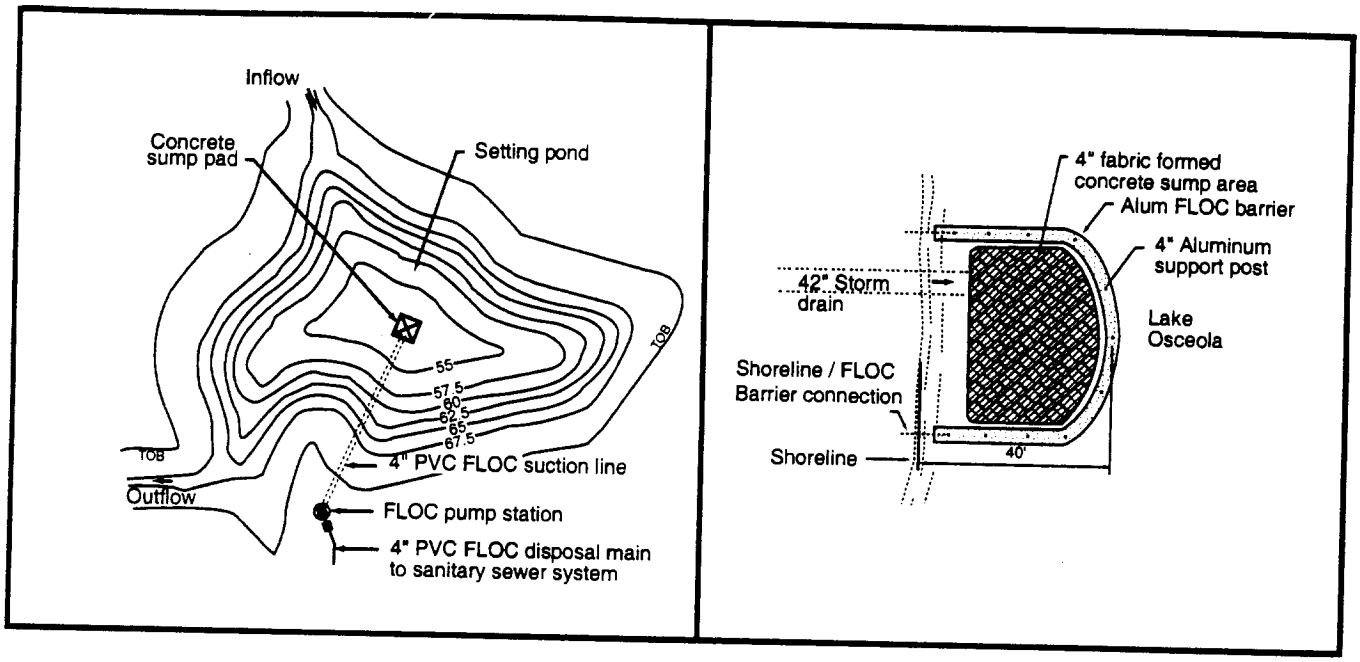

Figure 2. Typical schematics of floc collection and disposal systems.

tially lower than the predicted accumulation rate due to additional floc consolidation over time and incorporation of alum floc into the existing sediment.

- Construction costs for alum stormwater treatment systems are largely independent of the watershed area to be treated and depend primarily upon the number of outfalls to be retrofitted.
- In general, removal efficiencies obtained with alum stormwater treatment are roughly similar to removals obtained using a dry retention, and better than alternative stormwater management facilities.
- Alum treatment of stormwater runoff is often substantially less expensive than other stormwater treatment alternatives with respect to both initial capital costs and annual O&M costs.
- Several innovative designs have been developed for collection of alum floc in sump areas and containment areas, with floc disposal made to sanitary sewer or adjacent drying beds.

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An Eight-Step Approach to Implementing Stormwater Retrofitting

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What are retrofits and why are they important? In the quest for watershed protection and restoration, watershed professionals are constantly seeking new tools for controlling stormwater runoff and associated adverse impacts. Stormwater retrofits are among the most promising of these tools. Retrofits are structural stormwater management measures for urban watersheds designed to help lessen accelerated channel erosion, reduce pollutant loads, promote conditions for improved aquatic habitat, and correct past mistakes. Simply put, these best management practices (BMPs) are inserted in an urban landscape where limited stormwater controls existed.

Retrofits come in many shapes and sizes from large regional retention ponds that provide a variety of controls to small on-site facilities providing only water quality treatment for smaller storms. At least some kind of practice can be installed in almost any situation. But fiscal restraints, pollutant removal capability, and watershed capture area must all be carefully considered in any retrofit selection criteria.

Restoration versus Retrofitting

Stormwater retrofits should be applied along with other available watershed restoration strategies for reducing pollutants, restoring habitat and stabilizing stream morphology as part of a holistic watershed restoration program. While some professionals rightfully assert that true watershed restoration is not feasible, the term is applied here as simply a concerted strategy to install a functional native biological community in a stream, lake or river. Some of the many watershed restoration strategies include:

- Stabilizing stream channel morphology
- Improving aquatic habitat within urban streams
- Replacing or enhancing riparian cover along urban streams
- Promoting pollution prevention source controls within the watershed
- Recolonizing streams with native fish communities

Many, if not most, of these components should be planned in conjunction with an urban retrofit program. Without establishing a stable, predictable hydrologic water regime that regulates the volume, duration, frequency, and rate of flow, many of these other strategies may be disappointing failures. To successfully restore a stream's overall aquatic health, stormwater retrofitting is an essential element.

Table 1 presents a step-by-step approach to stormwater retrofitting developed by our Center for Watershed Protection staff over the past several years. An eight-step process is briefly discussed and several case studies from the author's experience emphasize particular points. At the conclusion of the eight-step process, two additional case studies are presented in more detail to illustrate some of the many real world challenges of implementing retrofit projects.

Step 1. Watershed Retrofit Site Inventory

The first step in getting retrofits "in the ground" is the process of locating and identifying where it is feasible and appropriate to put them. This involves a process of identifying as many potential sites as rapidly as possible. The best retrofit sites fit easily into the existing landscape, are located at or near major drainage or stormwater control facilities, and are easily accessible. For example, almost every urban area has some type of existing pond or other feature that might be adaptable for retrofitting. In many newer neighborhoods, dry stormwater detention facilities are present for flood control. In older neighborhoods there are often aesthetic ponds, or other water features which can make suitable retrofits. Table 2 lists some of the most likely spots for locating facilities, some common applications, and applicable case studies.

Usually the first step is completed in the office using available topographic mapping (a 5-ft contour interval is quite satisfactory), low altitude aerial photographs (where available), storm drain master plans, and land use maps (zoning or tax maps are best). Scouting for potential sites should follow the guidance discussed above in Table 2. Two important tasks need to be undertaken before ventur-

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Table 1. Basic Elements of a Stormwater Retrofitting Implementation Strategy

Step	Elements	Purpose
1	Preliminary Watershed Retrofit Site Inventory	First cut at identifying potential retrofit sites
2	Field Assessment of Potential Retrofit Sites	To verify that sites are feasible and appropriate
3	Prioritizing of Sites for Implementation	To set up a priority for implementing future sites
4	Public Involvement Process	To solicit comments and input from the public and adjacent residents on potential sites
5	Retrofit Design	To prepare construction drawings for specific facilities
6	Permitting	To obtain the necessary approvals and permits for specific facilities
7	Construction Administration and Inspections	To ensure that facilities are constructed properly in accordance with the design plans
8	Maintenance Plan	To ensure that facilities are adequately maintained

Table 2. Locations for Stormwater Retrofits

Location	Type of Retrofit	Case Study
Existing stormwater detention facilities	Usually retrofitted as a wet pond or stormwater wetland capable of multiple storm frequency manaagement	Wheaton Branch, Sligo Creek, Wheaton, MD – multi-cell wet pond with extended detention
Immediately upstream of existing road culverts	Often a wet pond, wetland, or extended detention facility capable of multiple storm frequency management	Epsilon Pond, Redland, MD – dry extended detention facility
Immediately below or adjacent to existing storm drain outfalls	Usually water quality-only practices such as sand filters, vegetative filters or other small storm treatment facilities	Long Quarter Branch, Towson, MD – gravel based wetland filter
Directly within urban drainage and flood control channels	Usually small-scale weirs or other flow attenuation devices to facilitate settling of solids within open channels	Indian Creek, College Park, MD – instream concrete weir flow attenuationi device
Highway rights-of-way and cloverleafs	Can be a variety of practices, but usually ponds or wetlands	Bear Gutter Creek, Route 22, Armonk, NY – combination wet pond and stormwater wetland
Within large open spaces, such as golf courses and parks	Can be a variety of practices, but usually ponds or wetlands capable of multiple storm frequency management	Meisner Ave Retrofit, Staten Island, New York City – micro-pool extended detention facility
Within or adjacent to large parking lots	Usually water quality-only facilities such as sand filters or other organic media filters (e.g., bioretention)	Kettering Subdivision, Prince Georges Co., MD – Bioretention practices

ing into the field. First, the drainage area to each potential retrofit site should be delineated and the potential surface area of the facility measured. The drainage area can be used to compute a capture ratio. Capture ratio is the percentage of the overall watershed that is being managed by all retrofit projects. The surface area is used to compute a preliminary storage volume of the facility. A shortcut storage volume consists of multiplying two-thirds of the facility surface area times an estimated depth ($2/3 \cdot SA \cdot d$). These two values can be used as a quick screening tool. In general, an effective retrofitting strategy must capture at least 50% of the watershed and a minimum target storage volume for watershed retrofit is approximately 1/2 inch per impervious acre within the watershed.

Step 2. Field Verification of Candidate Sites

Candidate retrofit sites from Step 1 are investigated in the field to verify that they are feasible. This field investigation involves a careful assessment of site specific information such as presence of sensitive environmental features, location of existing utilities, type of adjacent land uses, condition of receiving waters, construction and maintenance access opportunities, and most importantly, whether or not a desirable retrofit will actually work in the

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specified location. Usually a conceptual sketch is prepared and photographs are taken.

One study that incorporated the principles of this process was the Longwell Branch Stream Restoration Study conducted in Westminster, Maryland in 1994. The Longwell Branch watershed drains approximately two square miles of moderate-to-high density commercial and residential land. The area was built prior to stormwater management requirements, and consequently the stream system was suffering the typical urban impacts along much of its reach. This investigation utilized a retrofit inventory form which provided field investigators with specific information such as topography, property lines and ownership, storm drain outfall locations, drainage area, mapped utility locations, and other important site design features. This data helps field investigators decide if sites are to be retained or eliminated from further consideration. Longwell Branch retained approximately two-thirds of the identified sites. Those eliminated involved conflicts with utilities, wetlands or adjacent properties.

Step 3. Prioritize Sites for Implementation

Once sites have been located and determined to be feasible and practical, the next step is to set up a plan for future implementation. Even the best stormwater retrofitting programs have limited capital budgets for individual project design and construction. Therefore, it is prudent to have an implementation strategy based on a prescribed set of objectives. For example, in some watersheds, implementation may be based on a strategy of reducing pollutant loads to receiving waters where the priority of retrofitting might be to go after the "dirtiest" land uses first. But if the strategy is oriented more toward restoring stream channel morphology, priority retrofits are targeted to capture the largest drainage areas and provide the most storage. Whatever the restoration focus, it is useful to provide a scoring system that can be used to rank each retrofit site based on uniform criteria. A typical scoring system might include scores for the following items:

- Pollutant removal capability (storage provided and type of BMP)
- Stream channel protection capability (ability to control modest flow events)
- Cost of facility (design, construction and maintenance)
- Ability to implement the project (land ownership, construction access, permits)
- Potential for public benefit (education, location within a priority watershed, visible amenity, supports other pubic involvement initiatives)
- Percent of watershed capture

Step 4. Public Involvement Process

A successful project must involve the immediate neighbors who will be affected by the changed conditions. Nearly all retrofits require significant modifications to the existing environment. A dry detention pond, for example, is for some a very desirable area in the community. It is a place to walk the dog and only rarely is there any water in the facility. A wet pond or stormwater wetland retrofit, on the other hand, may have large expanses of water and may have highly variable water fluctuations. Adjacent owners sometimes resist these changes. In order to gain citizen acceptance of retrofits, affected persons must be involved in the process from the start and throughout the planning, design and implementation process. Citizens who are informed about the need for, and benefits of, retrofitting are more likely to accept projects.

Still, some citizens and citizen organizations will never support a particular project. This is why it is mandatory that there be an overall planning process which identifies projects early and allows citizen input before costly field surveys and engineering are performed. Projects that cannot satisfy citizen concerns may need to be dropped from further consideration.

This step of a good retrofit program must utilize a good public relations plan. Slide shows, or field trips to existing projects, can be powerful persuasions to skeptical citizens. Before any site goes forward to final design and permitting, it should be presented at least once to the public.

Step 5. Retrofit Design

The design process is for some, including this author, the most rewarding part of the process. Here, the concept is converted from a dream to a construction drawing. Design of retrofit projects incorporates the same elements as any other BMP project including: adequate hydrologic and hydraulic modeling, detailed topographic mapping, property line establishment, site grading, structural design, geotechnical investigations, erosion and sediment control design, construction phasing and staging to name a few. But BMP design for developing residential areas usually follows a prescribed design criteria (e.g., control of the 2-year storm or sizing for a specified water quality volume), whereas retrofit designers must work backwards from a set of existing site constraints to arrive at an acceptable stormwater control.

Sometimes this process yields facilities that are too small or ineffective, and therefore not practical for further consideration. One such project in Gaithersburg, MD, was recently proposed as a major stormwater wetland upstream from an existing road culvert to control a 1,000 acre watershed. The problem was that only 1/20th of an inch of total storage (.05") was obtainable. Clearly this facility would have been a maintenance nightmare and likely would have done little to remove pollutants or control downstream channel erosion. The City of Gaithersburg correctly decided not to pursue the project even though it had already retained a consultant and spent significant time and money on preliminary design.

The key to successful retrofit design is the ability to maximize pollutant removal and channel erosion protection

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while limiting the impacts to adjacent infrastructure, residents or other properties. Designers must consider issues like avoiding relocations of existing utilities, minimizing impacts on existing wetland and forest, maintaining existing floodplain elevations, complying with dam safety and dam hazard classification criteria, avoiding bad maintenance situations, and providing adequate construction and maintenance access to the site.

Step 6. Permitting

Permitting issues for retrofit projects often involve impacts on wetlands, forests and floodplain alterations. Many of these impacts are either unavoidable or necessary to achieve reasonable storage targets. Permitting agencies are primarily focused on ensuring that the impacts have been minimized to the extent practicable and that the benefits of the proposed project are clearly recognizable.

One recent project in New York City's Staten Island Bluebelt illustrates this point. A larger detention facility is being proposed for the Richmond Creek subwatershed to control a 400-acre headwater drainage area. The facility was initially conceived to provide a wet pond with wetland elements and extended detention of runoff from the 1-year storm. The facility is proposed within the Bluebelt park system where impacts to trees and wetlands were a major concern to park personnel and regulatory agencies.

Several alternative designs were presented that minimized wetland and forest impact while maximizing storage volume to provide downstream channel erosion protection. The real balancing act was to achieve enough storage to provide meaningful downstream channel protection and at the same time minimizing upstream impacts to a mature forest and wetland. The final acceptable solution consisted of a micro-pool wet pond with extended detention for the 1-inch rainfall event and a total disturbance limit of about a half acre.

Step 7. Construction Administration and Inspections

Like any major design project, proper construction inspection and administration is integral to a successful facility. This is especially important for retrofit projects. Retrofitting often involves construction of unique or unusual elements, such as flow splitters, underground sand filters, or stream diversions. These practices are unfamiliar to many contractors. Most publicly funded projects are awarded to the low bidder, who may be qualified to do the work but has never constructed projects of this nature before. Therefore, it is almost a necessity to retain the original retrofit designer or other qualified professional to answer contractor questions, approve shop drawings, conduct regular inspections, hold regular progress meetings, conduct construction testing, and maintain construction records.

Step 8. Maintenance Plan

Always the last element to be discussed, and often the least practiced component of a stormwater management program, maintenance is extremely important in retrofit situations. The reasons are simple. Most retrofits are undersized when compared to their new development counterparts, and space is at a premium in urban areas where many maintenance provisions such as access roads, stockpiling or staging areas are either absent or woefully undersized.

Designers (see Step 5) must balance maintenance access and storage volumes (for forebays, catch basins, and debris trapping areas) with water quality, flood control, and the other constraints discussed above. But in Step 8, the maintenance must be accomplished as designed.

Retrofit Case Studies

1. *Example of Retrofitting an Existing Stormwater Detention Facility Wheaton Branch, Montgomery County, MD*

The Wheaton Branch facility, located near Wheaton, MD, is arguably one of the best known modifications of a former dry detention facility, retrofitted to provide water quality and channel protection controls. The facility, constructed in 1990, drains an 800-acre watershed that is over 50% impervious. A unique design feature was the three-cell wet pond (constructed around an existing sanitary sewer trunk main) to provide water quality controls. Extended detention controls for the 1.5-inch rainfall event were incorporated for channel protection. The three-cell pond has a complex flow path for both baseflows and small stormflows to facilitate maximum settling of solids. Controls for larger storms (i.e., two - 100 year rainfall events) were balanced against upstream backwater constraints and dam safety considerations. Figure 1 illustrates the key operational and design elements of the project.

The first cell of the facility, or forebay provided almost a tenth of an inch of storage per impervious acre within the watershed (this is too small for most retrofits). A 25-foot wide access ramp with a level 30' by 30' pad was provided for future dredging. During the design phase, it was estimated that dredging of the forebay would be necessary every five years or so. The first cleanout of the forebay occurred in July 1997, a little over seven years after completion of the project.

The Wheaton Branch retrofit facility was part of the larger Sligo Creek watershed restoration project. Downstream habitat improvement and native fish restocking projects accompanied the retrofit and have proved very successful over their seven-year initial period. John Galli, (Galli and Schueler, 1992)and his colleague Jim Commins (1992) have published several reports and articles on the success of the stream restoration efforts in Wheaton Branch.

Some important design lessons are also illustrated by the Wheaton project. The existing hydraulic characteristics of the facility were first analyzed to assess the control originally provided. The original facility provided partial control of the 2-, 10-, and 100-year storm and safely passed

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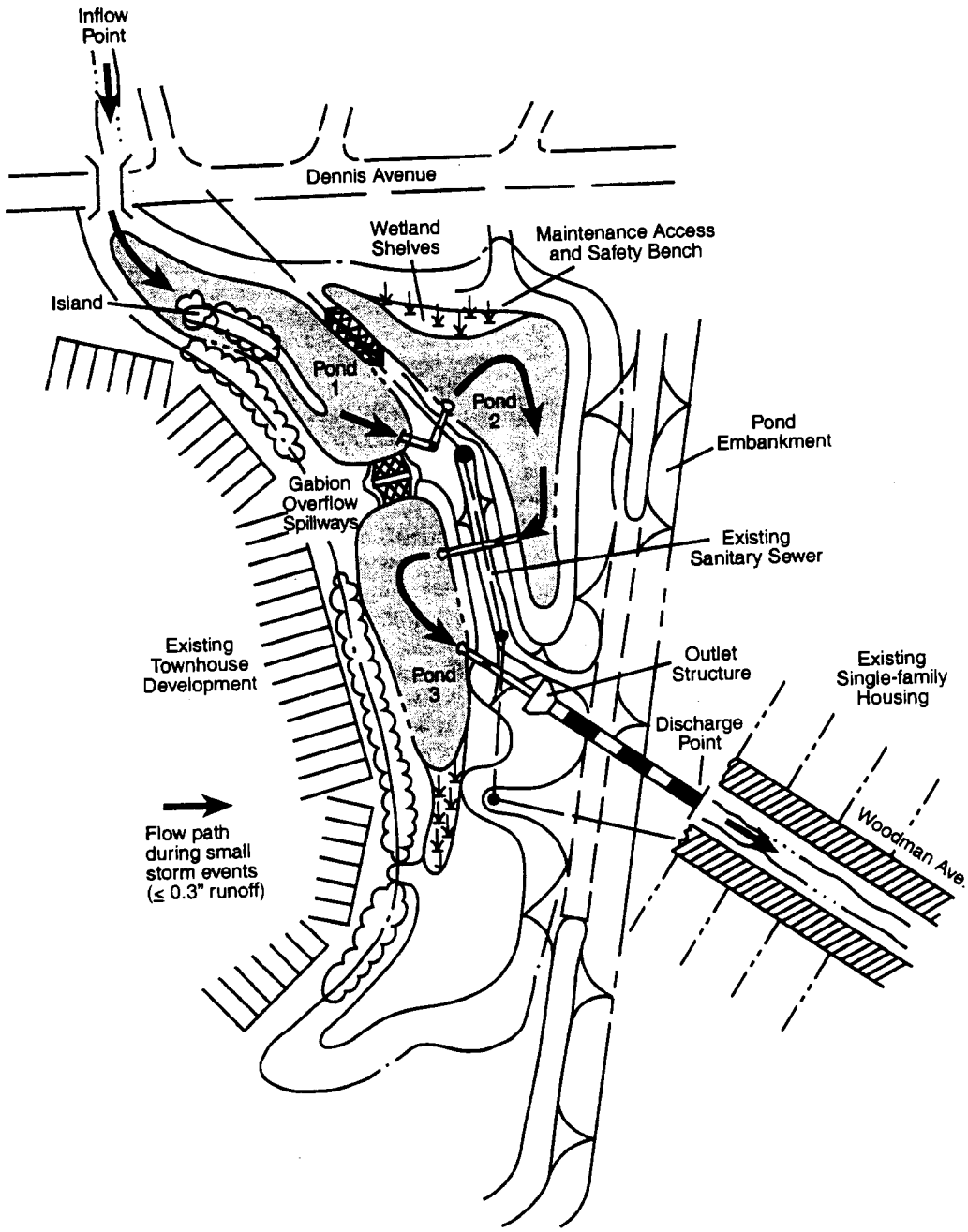

Figure 1. Wheaton Branch Retrofit key operation and design elements – Montgomery County, MD

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the probable maximum flood (PMF) through a massive emergency spillway. The retrofit required a balancing act to maximize water quality control, while maintaining enough control for larger storms to avoid impacting downstream houses or the 100-year floodplain.

Routing storms through the three-cell pond was extremely difficult due to the very low head conditions and the unusual backwater created by downstream ponds. The original pond bottom was excavated for much of the permanent pool storage (for pond and wetland components), the emergency spillway was modified to maintain passage of the PMF and the outlet control structure was completely overhauled.

All of these measures added up to quite an expensive project. The total cost for the facility, including engineering, construction, and construction inspection was approximately $800,000. Although this was certainly a healthy sum, it equates to approximately $640,000 per square mile ($1,000/acre) of drainage area. This is a third less than the typically quoted figure of approximately $1 million per square mile of drainage for average retrofitting (Karouna, 1989).

2. *Example of a Retrofit in a Highway Right-of-Way Bear Gutter Creek, Westchester County, NY*

The Bear Gutter Creek Retrofit is one of many projects recently designed to protect the Kensico Reservoir (one of the principal components of New York City's drinking water system) from impacts of stormwater runoff. The Bear Gutter watershed is approximately one square mile in area and drains mixed land uses, having approximately 30% impervious area, directly into the Kensico Reservoir. Note that this is an unfiltered drinking water system serving millions of New Yorkers. The retrofit is located immediately below a state road culvert and within the NY Route 22 right-of-way.

Interesting design features include a flow diversion weir at the downstream end of an existing large diameter road culvert which diverts baseflow and stormflow for up to the 1.5-inch rainfall event into a primary settling area. Storms larger than the 1.5-inch rainfall are diverted to a stabilized downstream channel below the facility. The primary settling chamber is sized for about 1/3-inch per impervious acre and has both a wet component and storm storage above the wet pool. An existing 1-1/2-acre emergent wetland, adjacent to the facility receives runoff as a polishing treatment below the primary settling chamber. See Figure 2 for an illustration of the facility and representation of design features.

The design criteria for the Bear Gutter Creek project (as well as all of the Kensico project) was to provide a facility

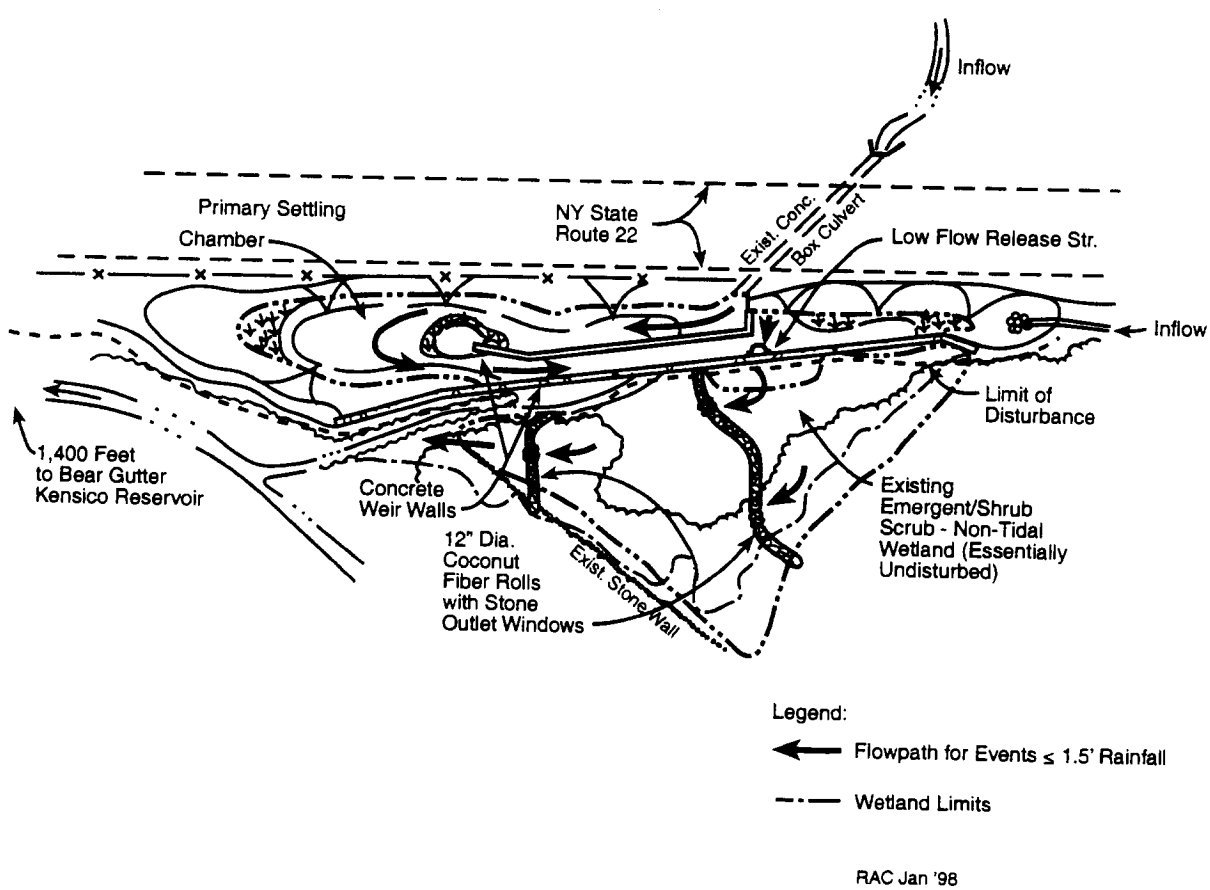

Figure 2. Bear Gutter Creek Retrofit – Illustration and representation of design features

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with the minimum storage volume necessary to maximize particulate settling, and provide long detention times to allow for fecal coliform dieoff. An original design concept called for siting the facility in the middle of the 1-1/2 acre wetland. Unfortunately very little space was available within the road right-of-way or anywhere else outside of the existing wetland. The solution was to use a flow diversion structure coupled with a concrete weir and baffle to maximize the length of the flow path within the primary settling chamber and then utilize the wetland as a "polishing" treatment. Coconut rolls were specified within the wetland to encourage additional detention for controls of larger storms.

Summary — Is Retrofitting Really That Complicated?

The answer to this question might seem elusive. Retrofitting can be a daunting task, and usually not an inexpensive one. The key to a successful local program is to follow a systematic and straightforward process toward implementation. The eight-step process presented above is certainly not the only way to get projects built. Some jurisdictions identify and construct pilot projects first and then expand a program from there. Others spend much more time on planning and public involvement. Whatever the focus, retrofitting is still more of an art than a science, and planners and designers who take an approach geared toward innovation will go a long way toward successfully planning, designing, and building stormwater retrofit projects.

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Identifying Wetland Restoration Opportunities in the Rouge River Watershed

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Abstract

This report discusses factors to be considered in identifying wetland restoration opportunities in an urban river watershed. The discussion is based on a recent study of wetland restoration opportunities in the Rouge River Watershed in southeast Michigan, funded by the United States Environmental Protection Agency through the Rouge River National Wet Weather Demonstration Project. Wetland ecosystem restoration or creation in urban settings presents certain unique challenges compared to similar projects in rural or undeveloped areas. Identifying appropriate sites for wetland restoration in urban settings requires special consideration of the unique characteristics of urban environments. Environmental challenges frequently encountered in urban settings include contaminated sources of water, contaminated soils, severe hydrologic conditions, unsuitable adjacent land uses, and severely disrupted plant and animal communities in existing wetlands. Social and economic issues that influence urban wetland restoration projects include opportunities for recreational uses of restored wetlands and environmental education in wetland areas. When wetland restoration projects account for these factors, remarkable wetland ecosystems can be restored in urban areas.

Introduction

Wetland restoration has become a widely used part of many river restoration efforts. Wetland ecosystems provide many functions to a watershed, such as fish and wildlife habitat, water quality improvement, flood water storage, and passive recreation. In an urban environment, these wetland functions have usually been severely impacted over the years so there are benefits to be gained by restoring wetland habitat as part of any river restoration effort.

While restoration of wetland habitat has been an active part of resource management for many years, most wetland restoration efforts have been focused in rural areas as part of wildlife management programs. Organizations such as the US Fish and Wildlife Service, state wildlife departments, and private conservation organizations such as Ducks Unlimited have restored thousands of wetland acres over the years. The techniques for wetland restoration in agricultural environments have been developed over many years and are based on many experimental approaches. The state of the art is such that wetlands can be restored to provide habitat for specific plant and animal communities. Several guidebooks are available for the habitat requirements of certain wildlife species.

Since the 1970s, research has been conducted on the design and construction of wetlands that provide functions other than fish and wildlife habitat. The majority of this research has focused on the use of wetlands for wastewater treatment, but there have also been some notable efforts to design wetlands for the specific purpose of abating nonpoint source pollution. The wastewater treatment field has evolved to the point where design manuals have been published detailing the design and construction of several different types of treatment wetlands. Supplementing these manuals is a series of guidebooks describing design guidelines for stormwater treatment wetlands (Schueler, 1992).

While the information and techniques developed for wetland restoration in wildlife management and nonpoint source pollution treatment is valuable, the application of this information to urban wetland restoration projects needs to be critically evaluated. The urban ecosystem is limited by physical, chemical, and biological characteristics that influence the types of wetland ecosystems that can be practically restored. The urban ecosystem is sufficiently different from undisturbed ecosystems that design manuals need to be modified to reflect the urban environment. The following information is meant to highlight some of the key aspects of the urban ecosystem that should be considered when undertaking wetland restoration projects in urban settings.

Physical and Chemical Factors

Frequency and duration of flooding. The hydrologic character of urban rivers and streams frequently consists of widely varying fluctuations in flow rate and water level and frequent flooding. While wetland habitats are also frequently flooded, the frequency and duration of flooding in an ur-

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ban setting is usually greater than in undisturbed rivers and needs to be considered. The flooding frequency of certain urban rivers may be severe enough to preclude the establishment of forested wetlands. Forested wetlands in northern areas of the United States require a period of relatively low water levels in order to survive.

The Rouge River is a river ecosystem that is characterized by a diversity of hydrologic conditions with a diversity of frequency and duration of flooding. Some reaches of the river experience flooding out of the river channel an average of eight times a year predominantly in the early spring. Water level fluctuations accompanied by these flood events typically represent a rise in river water level of six feet. Wetland restoration sites that were identified in the Lower Rouge River were all within the floodplain, but at locations where the water level fluctuations were typically less than three feet. The decision to locate wetland restoration sites in areas with moderate flood elevations was made in an effort to reduce the adverse impact of severe frequent flooding on waterfowl nesting success. All of the wetland restoration designs accounted for the frequency and duration of flooding and were designed to allow flood waters to inundate restored wetlands, thus imitating the relationship between riparian wetlands and flooding conditions in the river.

Water level fluctuations. In locating wetland restoration sites, one should consider the magnitude of water level fluctuations. Wetland habitat that is flooded by three to four feet of water may be detrimental by flooding wildlife populations, especially if the flood occurs during the waterfowl nesting season. Water level fluctuations in wetlands have been recognized by wildlife biologists as important aspects of wetland management, and they frequently manage wetlands by draining the basin to encourage plant diversity. The wetland restoration sites in the Rouge River were expected to receive stormwater runoff during periods of precipitation, but the character of the watershed draining into the wetlands was such that the water level in the wetland would not recede between storm events. While permanent open water wetlands are important to certain species of wildlife, the wetland inventory of Wayne County indicated that open water wetlands were relatively common in the vicinity of the wetland restoration site. The design of the wetland restoration was, therefore, intended to produce a wetland that would have moderate water level fluctuations. This was accomplished by designing inlet diversion structures that would divert only a portion of each storm event into the wetland, thus allowing for minor water level fluctuations typical of undisturbed wetlands along the river. Simplified hydrologic models of the water balance for the wetland were used to determine the volume of water to be diverted into the wetland with each storm event.

Flow rate. Wetland restoration projects should minimize the flow rate of water flowing into the wetland because of the damage caused by high velocity to habitat and fish and wildlife populations. Receiving streams in urban settings are frequently subjected to water flowing at high velocity, which causes significant adverse impacts. Wetlands may be significantly impacted in a similar manner unless measures are taken to mitigate the potential impacts. One effective technique that has been used in the Rouge River is to provide a regulated inlet structure that restricts the amount of water that flows into the wetland. Bypass structures are used to protect the wetland from severe velocity and the accompanying damage to fish and wildlife populations.

Water quality. Wetland restoration efforts can be severely constrained in urban efforts unless the water quality of urban runoff is considered. While wetland habitats have been recognized as potential nutrient sinks, wetlands can also be damaged by severe nutrient loading. Several approaches were used to manage the potential adverse impact of excessive nutrient loading. First, the wetland restoration sites in the Rouge River are protected by sedimentation basins prior to the wetland that remove excessive sediment and nutrient loading. The basins were designed with a restricted outlet that allows for floating material and sediment to be retained in the sediment basin. The effect of such sediment basins is to limit sediment load to the wetland and to provide a location where sediment can be removed during routine maintenance and operation. The second approach was to design wetland restoration projects that planned for plant communities adapted to high nutrient loading. In practice this meant that there was a predominance of emergent wetlands and a minimum of open water and submergent wetland types.

Bioaccumulative contaminants. Several recent reports have shown that levels of bioaccumulative contaminants, such as heavy metals, are higher in fish and wildlife in stormwater ponds compared to populations in wetlands not exposed to urban stormwater. The Canadian Wildlife Service and Environment Canada (Wren, et al.,1997) reported that persistent chemicals bioaccumulate in sediments, water, and wildlife and, in some locations, the chemical concentrations exceeded the Ontario Sediment Quality guidelines for low effect level for aquatic animals. The report indicated that definitive estimates of exposure and effects are required to clearly understand the risk to wildlife populations. Similarly, in a study of fish species in stormwater treatment ponds in Florida, (Campbell 1994) reported that red ear sunfish had significantly higher levels of cadmium, nickel, copper, lead, and zinc compared to fish from control ponds. Bluegill and largemouth bass collected from stormwater ponds also had significantly higher levels of heavy metals compared to control ponds.

Wetland restoration projects in the Rouge River accounted for the potential accumulation of contaminants by incorporating several protective measures. Wetland sites with heavy industrial uses in the watershed were avoided. When wetland sites with runoff from commercial areas were selected, sediment basins were designed to trap contaminants bound to sediment particles prior to the runoff entering the wetland. Restored wetland habitats were also designed to minimize the potential for incidental ingestion of

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contaminated sediment by fish and wildlife species by limiting the area of open water available for fish and waterfowl feeding.

Biological Factors

Target wetland communities. Wetland restoration efforts should be based on the type of wetland habitats that contribute to the restoration of a diverse river ecosystem. In certain river ecosystems, emergent wetland habitats designed to abate nonpoint pollution sources are important, while in other regions it may be more appropriate to restore riparian forests that support wildlife species and stabilize river banks. The type of wetland to be restored should be determined prior to searching for potential sites and should be based on an open review of wetland resources and natural resource restoration goals for the river ecosystem.

Maps of the wetlands of the Rouge River indicated that approximately 70% of the wetlands in the watershed have been lost. The analysis also showed that the most common type of remaining wetland is forested wetland habitat concentrated along the river corridor. A review of vegetation maps of the vegetation prior to European settlement indicated that while forested wetlands were present, there were also extensive areas of emergent wetlands in the watershed. Emergent wetlands are recognized as being effective in abating nonpoint sources of pollution and are important areas for fish and wildlife populations. Based on the historical vegetation data, and the goal to manage nonpoint source pollution of the river, a decision was made to restore emergent wetland habitats to the greatest extent possible.

Existing habitat. Whenever possible wetland restoration sites should be connected or linked to existing wetland habitat. An important benefit of this approach is that plant and wildlife populations in the existing wetlands have an opportunity to migrate into the new wetland habitat. The existing wetlands maps for the Rouge River watershed were used to identify existing habitat along the river corridor. Wetland maps were then compared to maps of hydric soils to indicate the potential for wetland restoration. Each existing wetland was visited and assessed for wildlife use and condition of existing habitat. The result of this analysis of existing wetland habitat and quality was that wetland sites were restored adjacent to forested wetlands that had viable populations of amphibians, reptiles, and waterfowl.

Nuisance plant, fish, and wildlife species. Wetland restoration projects are frequently subjected to significant damage by nuisance plant, fish, and wildlife species. Carp can destroy plantings of submerged and emergent plant species and cause severe turbidity due to feeding activity. The resultant turbidity can suppress development of submerged aquatic plant communities. Grazing of wetland plants by geese can be severe and has in some cases resulted in complete removal of plants in newly planted wetlands. Nuisance plant species include invasive wetland plant species, such as purple loosestrife and reed canary grass, which can dominate a wetland, preventing the development of a diverse plant community. On the Rouge River, carp have been controlled at wetland restoration sites by limiting the connection of the wetland to the river. When it was necessary to connect wetlands to the river, a chain link fence was installed across the connection to the river to exclude adult carp. However, carp were introduced into several restoration sites during a flood and manual methods of control have been necessary. Canadian geese damage has been managed by stringing rope with flags across the wetland at 50-foot intervals until the wetland plant community was established. The ropes seem to disturb geese from landing on the wetland surface thus preventing grazing in the wetland. After the plant community was stabilized, the ropes were removed.

Ecological traps. Ecological traps are areas of attractive habitat that represents an unsafe condition for plant or animal populations (Gates and Geysel, 1978). Species mortality in ecological traps frequently exceeds reproduction due to the hazards represented in the habitat. A stormwater treatment basin with steep side slopes and permanent open water is an example of an ecological trap. Waterfowl nests located within the storage elevation of the retention basin can be destroyed during precipitation events and result in adverse impacts to reproduction of the species. Wetland restoration in urban rivers needs to be planned in such a way that ecological traps are not created.

Ecological traps were prevented in the Rouge River wetland restoration areas by linking the new wetlands to existing habitat that offered an opportunity for a sustainable population of plant and animal species. Buffer areas of natural vegetation were created around all new wetlands to provide upland habitat for wetland species that require upland habitat for part of their life cycles. Wetland restoration sites surrounded by urban or industrial property were avoided. Finally, all restored wetland areas were linked in one manner or another with wildlife corridors to allow migration of wildlife species.

Socioeconomic Factors

Environmental education. Wetland restoration projects represent an opportunity to provide outdoor classrooms to local schools and recreation departments. Natural habitat in urban settings are rare and if a restored wetland area is planned near a school, the opportunity to explore cooperative arrangements with the school may be worthwhile. The wetland restoration sites in the Rouge River have been used by the science classes studying aquatic ecology and soil chemistry, as well as art and creative writing classes. The wetland is located within one mile of the high school and is being used by additional classes and programs each year.

Passive recreation. The emphasis of wetland restoration is the renewal of wetland functions, such as fish and wildlife habitat, water quality protection, or flood water storage. When the opportunity arises, however, significant public benefits can be realized if passive recreation is in-

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tegrated into the wetland restoration project. In the Rouge River project, we found that wetlands that had a trail or public access site were used by residents in the neighborhood. During monitoring visits, the residents would share with us their experiences in the wetland regarding wildlife observation or water level fluctuations. Signage at future wetland restoration projects will assist residents in understanding the restoration project and the benefits gained from restoring wetlands.

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Taking Root: Sowing and Harvesting the Seeds of Public Involvement and Education

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The Rouge River, a tributary to the Detroit River, in southeast Michigan, has been well documented as a significant source of pollution to the Great Lakes System. The Rouge River Watershed spans approximately 467 square miles involving 48 communities, three counties, and over 1.5 million residents. The eastern portion of the watershed consists of much of the old industrial areas of Detroit and Dearborn. The western and northern portions consist of newer suburban communities and areas under heavy development pressure.

This paper discusses the products, programs, and partnerships used by the Public Involvement team of the Rouge River National Wet Weather Demonstration Project (Rouge Project) to: 1) first increase watershed awareness in Rouge River watershed residents; 2) educate them about the pollution sources to the Rouge River; and then 3) involve them in restoration of the Rouge River by showing them that small changes in their daily activities can help restore the river.

Even before the inception of the Rouge Project four years ago, it was clear that a comprehensive public involvement and education program was necessary to support Rouge River restoration activities. A survey of watershed residents in 1994 determined that, while few people viewed the Rouge River as a viable resource because of its pollution, the majority broadly supported efforts to improve its quality. The survey determined that a grassroots approach coupled with a top down strategy was needed. A public outreach strategy, based on the survey, used the philosophy that communication with Rouge River watershed stakeholders must be continual, consistent, truthful and always two-way.[1]

The strategy identified seven stakeholder groups that must be educated: the general public, local government officials, industry and business, environmental and community groups, the technical community, the media, and schools.

So, for instance, as local officials were being educated about the Rouge River's problems and possible solutions, the Public Involvement team was developing easy-to-read fact sheets, brochures and posters for the general public. At the same time, pilot pollution prevention programs in three watershed neighborhoods and two business areas were developed to fashion a watershed-wide program and fine-tune appropriate messages for the general public and businesses.

It became clear that several themes had to be used to educate watershed residents. First of all, stakeholders had to learn that they lived in a watershed containing 48 communities and parts of three counties. This is a diverse watershed where land uses range from undeveloped land and farms on its western edge to heavily urbanized industrial sections on its eastern boundary. Household incomes are just as diverse: in Bloomfield Hills, in Oakland County, the average household income is $150,001 while Highland Park, in Wayne County, has an average household income of $9,805. Furthermore, stakeholders needed to understand that the pristine, rural tributary in northern Oakland County was part of the same watershed as the brownish, urban river flowing past the massive Ford Rouge Plant complex in Dearborn.

In addition, since many of those surveyed thought the pollution in the Rouge River was from industrial sources, a public education campaign had to explain storm water and non-point source pollution, and their impacts on the Rouge River. Finally, the campaign had to explain to all stakeholders their personal role in Rouge River restoration efforts.

Products

A number of products were created over the past two years to accomplish our goals. They include:

Activity Book: This 12-page booklet was designed for elementary school students. It contains word searches,

[1]For information about Rouge Project products or stategy, contact Josephine A. Powell, Wayne County (Michigan) Department of Environment, Detroit, MI 48226 Telephone: 313-224-3620.

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crossword puzzles and connect-the-dot pictures that entertain while delivering a message about non-point source pollution, watershed awareness and personal responsibility. Over 30,000 books have been delivered to area schools through requests from teachers in six targeted areas in the watershed, to schools that participate in the Friends of the Rouge Education Project and to teachers who attend education conferences. In addition, 10,000 Rouge River posters, featuring wildlife that live in or near the river, were distributed to area schools. The books and posters are also used as a handout at watershed fairs and other community events.

Placemats: Restaurants in the watershed were approached about using a Rouge Project placemat that featured a drawing by a fourth-grade student who won a Friends of the Rouge poster contest. The back of the placemat listed Rouge Friendly pollution prevention tips. Close to 50,000 placemats were distributed to watershed restaurants and were enormously popular, according to a survey of restaurant owners. A second placemat, recently developed, features watershed education, and promotes recreational activities in the Rouge River. Over 50,000 placemats have been used by area restaurants with another 50,000 on order. Currently, the Rouge Project is negotiating with a popular fast-food restaurant to feature a Rouge River placemat in its restaurants during Earth Day week in April, 1998.

Theater Advertising: Three public service announcements were developed for a Rouge River awareness campaign that ran on theater movie screens prior to the featured film, for five months. The first one promoted Rouge Rescue, an annual river clean-up event sponsored by the well-respected grassroots organization, Friends of the Rouge; the second featured a Great Blue Heron as an example of wildlife that lives along the Rouge River; and the third featured Rouge Friendly household tips.

Newspaper Insert: A local community newspaper chain agreed to re-visit the Rouge River for a 10-year update to a section it produced in 1986 entitled " Our River: We discovered it; We settled along its banks; We built homes, farms and factories; And slowly, steadily we began to kill it." The update was a 12-page award-winning insert that reached 160,000 households in 12 communities in the Rouge River Watershed. Entitled "Changing Currents," the section documented the positive changes occurring in the Rouge River Watershed; discussed pollution prevention and Rouge-friendly behavior; and included a watershed map that was later reproduced as a poster.

Door Hanger: To augment storm drain stencilled messages organized by Friends of the Rouge, a door hanger in the form of a fish was developed as a leave-behind information piece for neighborhood residents. Text on the door hanger explains storm drain stenciling activities and lists Rouge Friendly home care activities. To date, over 10,000 door hangers have been distributed.

In addition, an ongoing media campaign has been instrumental in educating the public that they live in the Rouge River Watershed; that everyone is part of the pollution problems in the Rouge and can be part of the solution; and that with everyone's help, the Rouge River can be restored.

Programs

Pollution prevention programs were pilot-tested in three watershed neighborhoods and two business areas to fine-tune public information materials and program elements. They are:

The Rouge Friendly Neighborhood Program: This pollution prevention program was piloted in watershed neighborhoods in a number of areas of the watershed to promote education, river stewardship and storm drain stenciling activities. One pilot area, Brightmoor, is a Detroit neighborhood where the Main Rouge flows through a large park that borders the area. This densely populated, urban neighborhood is bisected by a business strip of predominately auto service businesses and has been plagued by illegal dumping and disinvestment. The local neighborhood organization is very active in the annual Rouge Rescue event sponsored by Friends of the Rouge.

Since the inception of the Rouge Friendly Neighborhood Program, the residents organized a business association that holds monthly meetings attended by city officials from the police department, the department of public works and the local neighborhood city hall and officials from the county environmental department. Environmental issues are a standing agenda item at the meetings and several businesses have qualified to be Rouge Friendly Businesses, which will be discussed below. The residents, in conjunction with Friends of the Rouge, hold a spring and fall Rouge Rescues, and in conjunction with the city parks department, received a Rouge Project community grant to remove several log jams from the river, stabilize river banks, plant wildflowers and native grasses and restore nature trails in the park.

In another pilot neighborhood in a more upscale area, residents have adopted a wetlands that adjoins their subdivision, participated in storm drain stenciling activities and promoted Rouge Friendly lawn and garden tips.

The Rouge Friendly Business Program: The Rouge Friendly Business Program is a companion pollution prevention program to the neighborhood program and was modeled after a similar program in Bellevue, Washington. It is aimed at educating small-to-mid-sized businesses to recognize that they can positively impact the Rouge River by making small changes to daily business practices. Since auto-related businesses are very common in the Rouge River Watershed, an automotive services roundtable, made up of representatives of automotive service associations and the local chamber of commerce and a businessman, was convened. The group met for nearly a year to review draft materials, make suggestions about program promotion and help mold the program before it was implemented. In addition, the industry representatives promoted the program in their publications and helped recruit businesses

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to participate in the program. Currently, there are 20 Rouge Friendly businesses. In addition, a pilot partnership has been formed with the state Environmental Assistance Division and RETAP program to aim a similar effort at the metal finishers industry. Currently in the planning stage is the creation of similar roundtables for food services and the construction services.

Based on what was learned from the neighborhood and business pilots, the Rouge Project public involvement team has fine-tuned its watershed awareness messages and is now promoting them via slogans such as:

- Use your head, you live in a watershed
- Storm drains aren't garbage cans
- When it comes to pollution, every home is waterfront property
- Everyone is part of the problem and needs to be part of the solution
- Simple changes can make big differences

These messages were incorporated into new tools, like magnets and brochures, that were developed in the past several months. A display, created to stress these messages, includes a watershed map and pictures of Rouge Friendly activities and tips. In the past year, the display has been exhibited at 41 watershed events where public involvement staff made contacts with approximately 27,000 people.

Partnerships

In order to give our messages momentum, a Rouge Public Involvement Team was established that has formed numerous partnerships with many stakeholder groups. Those partners include:

- Watershed townships and cities that have begun to use our "Storm Drains" display at their events.
- A resource recovery authority that has coupled its backyard composting/ yard waste reduction messages with Rouge River Watershed water quality information and pollution prevention techniques.
- A newspaper chain that writes about Rouge restoration and awareness activities almost weekly. The newspaper chain also features a Rouge River guest column written monthly by watershed stakeholders.
- Neighborhood organizations that distribute Rouge materials and spearhead stewardship activities for their section of the Rouge.
- Business associations that helped develop Rouge Friendly business materials and then spread the word about Rouge Friendly activities to their membership. Business owners were also instrumental in restoration activities by making Rouge Friendly activities part of their daily business practices.
- Friends of the Rouge, a grassroots organization, that has spread a stewardship message through several programs, many in conjunction with the Rouge Project. The Friends of the Rouge Education Project teaches water quality testing in 100 watershed schools and oversees a student sampling day of the Rouge River every May. Students are encouraged to create displays and videos for a Student Congress later in the month. The organization also sponsors an annual Rouge Rescue clean-up in several watershed communities and has begun to organize a second clean-up in the fall.
- The Rouge River Remedial Action Plan Advisory Council, composed of representatives from local government, business, the general public and non-profit organizations, that helped create a Rouge River Recreational Guide map to be distributed in the spring of 1998. In addition, Rouge Project public involvement staff participate in various subcommittees of the RRAC and facilitate public education efforts of these committees.
- The League of Women Voters which awarded a wetlands education fellowship to a member who is a resident of a Rouge Friendly neighborhood. She has spoken several times locally about the benefits of wetlands and made presentations on behalf of the Rouge Project. League of Women Voters members have also helped distribute Rouge River educational materials.
- The Greening of Detroit which co-sponsored an Arbor Day event with the Rouge Project for 200 elementary school children incorporating a tree planting event with a demonstration of Rouge Friendly tips.
- Oakland Community College, where an environmental studies teacher has made the Rouge River a regular part of her lesson plans. She also has given her students extra credit if they perform volunteer activities relating to Rouge River awareness.

Although quantifying our success has been difficult, we do have a baseline survey that showed that people are committed to doing what they can do to improve the Rouge River and a future survey will evaluate if the awareness of watershed residents has increased since the inception the public involvement activities. In addition, efforts are underway to better record and report information such as areas stenciled, stream miles adopted, the number of Rouge Rescue sites, the number of participating Rouge Education Project schools, and other volunteer efforts and opportunities as a way of measuring success and documenting compliance with the public education requirements of Michigan's voluntary storm water general permit.

Meanwhile, we are heartened by the following:

- The popularity of the Rouge River Activity Book; it has been a big hit at local teachers' conferences as well as in watershed schools.
- The number of personal contacts we've made at area community events.

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- The popularity of the two Rouge River placemats with restaurant owners and their customers.
- The interest of local elected officials and municipal employees who ask for their own copy of our display, and their willingness to distribute Rouge materials.
- The use of our ideas and products by other watershed groups nationwide.
- The increased use of the Rouge River for recreational purposes as people learn of its potential.

Research Findings, Recommendations and Lessons Learned

Listed below are various recommendations that the Rouge Project public involvement team would encourage others to consider when developing public education and involvement materials or programs. These recommendations are based on the research findings or advice received from others, including a Watershed Management Peer Review.

- To effectively inform and involve watershed residents, four central themes should be kept in mind: (1) establishing and maintaining a two-way flow of information with key stakeholder groups; (2) establishing or expanding educational mechanisms aimed at children; (3) building partnerships and utilizing communication networks and resources of existing organizations; and (4) devising ways to use the various public media to best advantage.
- Utilize third parties. As much as possible, try to have information come from sources such as non-governmental organizations or environmental groups, the state environmental regulatory agency, EPA, and recognized experts from the academic/scientific community.
- Peer-to-Peer communication/education is also vital. People respond positively to suggested, behavioral changes if they are being promoted by individuals whom they know and trust or regard as their peers.
- Balance public information materials with public involvement programs. Merely providing information is not sufficient. Programs and activities must be devised and implemented that actually involve citizens and other stakeholder groups in restoration efforts.
- Recognize and appreciate that each target audience, or stakeholder group, will need to be taken through a "communications continuum" of Awareness, Understanding, Involvement and Action.
- Scale is important. To the extent practicable, efforts should be directed to the local level. Implementing education and involvement programs at the right scale and bringing together stakeholders who share a common resource is critical to the success of public education programs. The slogan "Think Globally, Act Locally" surely applies.
- Avoid artificial barriers. Involvement opportunities (particularly in the early stages) should not be limited to only a few activities. To the extent practicable, individuals should be able to choose among a menu of activities or involvement opportunities; and within any given involvement program, flexibility should be offered to allow adaptation to local conditions and personal preferences.
- Lead/educate by example. Educational efforts should not be limited to the general public or community stakeholder groups. Local government agencies must recognize the impact that their own municipal operations have on pollution prevention. Internal staff awareness and education programs should be developed in the areas of equipment and vehicle maintenance, golf course and public lands maintenance, facilities maintenance, and land use planning, to name a few.
- Keep it fun and celebrate small successes. Remember, change doesn't happen overnight: it happens in bits and pieces.

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The Tollgate Wetland's Educational Experience

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Ingham County Drain Commissioner
Mason, Michigan

The Tollgate Drainage District Sewer Separation Project (TDDSP) involves a watershed of 234 acres, 550 residential homes, ten commercial properties, over 500 apartment units, and four governmental agencies. The Project involved the separation of a combined sewer system, and creation of a wetland detention basin. In addition to its stormwater detention uses, the wetland serves as a wildlife refuge, learning center, and, a local point of public outreach to bring the community together.

Introduction

Community organizers have long proclaimed the benefit of involving stakeholders in all projects in order to ensure their success. Residents of the Tollgate Drainage District/watershed were incorporated early in the 'planning process. Methods by which they were incorporated varied. Guiding principles for educating residents and encouraging their involvement were that 1) an investment in attitude and behavior change is a good investment, and 2) retrofitting water resource protection into urban environments will be more successful when accompanied by education and public outreach.

This project included a concept development and planning phase which suggested 1) the implementation of an education and public outreach program during project construction, and 2) the establishment of a long-term assessment and maintenance phase. During the planning process, public outreach was accomplished mainly through mailings and public meetings These meetings minimized negative feelings among the stakeholders.

Conflicts existed between the City of Lansing (Michigan) and Lansing Township due to the City's mandate that the Township separate the stormwater and sanitary water, as part of the City's sewer separation project. The traditional method for disposing of stormwater would be to take it to the nearest river. This cost was estimated and determined to be excessive, and likely to put the Township in financial difficulty. But once the City struck an agreement with the Township to take their sanitary sewer water, the Township had to develop some outlet for their stormwater.

TDDSSP employed an unusual method of dealing with the stormwater. Rather than piping it to the river, the water was retained in the neighborhood and a wetland complex was designed to treat the water on site. This method cost approximately one-third of the traditional method of disposing of stormwater, a savings of $15 million or more.

However, this solution would work only if the residents, or "owners" of the project, and primary stakeholders, changed their social behavior to reduce pollution loading, and act as owners of the wetland complex. In order to bring about changes in behaviors, the residents were given information about how their actions affected the outcomes of the stormwater management system and its costs.

For purposes of the remainder of this paper, the outreach/education approach is divided into two phases. The first is "during construction," or short-term, and the second is "after construction," or long-term. Figure 1 shows the relationship between education/public outreach and construction, and accomodates its impact on the project during the process, and on long-term assessment and maintenance.

Goals of Education and Public Outreach

The public outreach and education goals "during construction" were two-fold. The first was to facilitate commu-

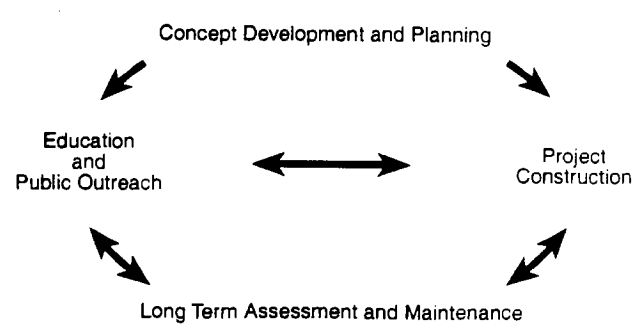

Figure 1. Diagram of Tollgate Drain Project.

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nity participation and ownership of the multi-objective project. The second was to change attitudes about the project. To achieve these goals, it was necessary to study the demographics of the resident population, and to learn about their attitudes toward portions of the project.

Three surveys were designed. The first survey indicated long-term residency, conservative beliefs and distrust of public officials. Residents also expressed opposition to change, and did not consider themselves to be environmentalists. It was also learned that their knowledge of environmental issues was limited. For example, when asked if they knew what non-point source pollution is, only 17% of respondents said yes (Figure 2). The survey also indicated that over 65% of respondents fertilized their lawns with a regular program, and half of those were applied by professional lawn care companies. Our baseline data, collected via water quality tests performed prior to construction, showed excessive amounts of fertilizer in the stormwater.

We concluded that public outreach had to be conducted, for both short and long-term outcomes, to bring about attitude change and a stronger sense of environmental stewardship. A number of approaches were employed to make this happen.

Education and Public Outreach During Construction

Public meetings were organized during both the planning and construction processes. These meetings served two main purposes. They assisted in informing the residents about the oncoming project, and they gave stakeholders an opportunity to vent their anxieties.

A neighborhood network was established to bring the community together. The Tollgate District was divided into 11 different sections, and block captains were chosen. Block captains formed phone trees which were utilized to disseminate updates on construction activity, organize neighborhood meetings and block parties. These meetings and block parties were used as forums to educate residents.

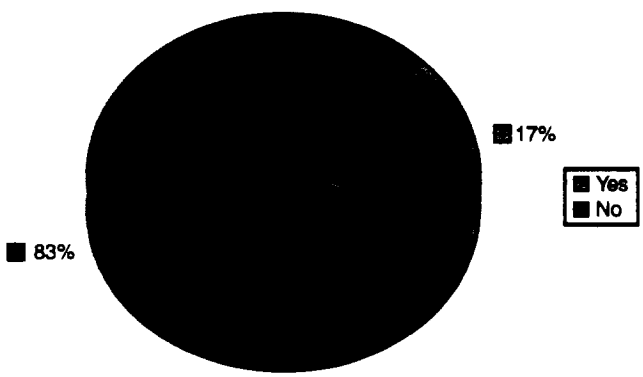

Figure 2. Tollgate survey response to the question, "Do you know what non-point source pollution is?"

An on-site office was opened in an apartment in the neighborhood. It was staffed with field inspectors; a hotline was established, and information was distributed from it. Two recent college graduates were hired to work there. They "leafleted" the neighborhood each week, with updates on street closures, paving schedules, and other current project information. They became a positive project presence and were available to residents to assist in aspects of daily living that were impaired by the construction, and answered questions as they arose.

Tours were offered to residents, at their convenience, and explanations were offered. Twenty-four hour responses were available for all problems during the construction phase.

The cost of this during construction outreach was a fraction of the total project. This up-front investment in outreach achieved our goals in a cost-effective manner, as Figure 3 explains.

Besides the pre-construction survey already mentioned, surveys were also done "during construction" and "after construction". The first survey was conducted door-to-door and of the possible 554 single-family homeowner respondents, 549 completed the survey of demographics, behaviors and attitudes. The second and third surveys also concentrated on attitudes and behaviors. Attitudes toward the project changed from 35% positive before construction began to 81% positive after construction was completed.

The public outreach will continue, long term, and will contain several component parts. First, further surveying will be conducted. Surveys will determine the success of public outreach in changing social behaviors and attitudes.

Second, water quality testing will occur on a regular schedule over the next seven years. Should public outreach be successful, there should be a reduction in pollution loading. Stormwater quality should improve, as attitudes and behaviors change. Data recovered from water monitoring will become part of a feedback loop designed to provide information and education about successes and failures.

Third, staff members of the drain commission have been trained in all aspects of this project, and are prepared to respond to stakeholders in ways that encourage continued partnership in the project. Maintenance needs of the wetlands and the storm/sewer system should decline as behaviors change. Should behaviors toward pollution loading not change, maintenance costs will ascend, which we do not want to happen.

Fourth, education will be ongoing and will not be limited to this watershed. It will include a variety of participants and programs. Partnerships have been formed with local environmental groups such as Urban Options, which has

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• Staffing	$27,000
• Field Office	$2400
• Supplies	$1000
• Hotline	$300
Total	$30,700

Less than 1/3 of 1% of the Total Project Budget

Figure 3. Education and public outreach investment (during construction).

agreed to 1) send representatives to neighborhood block parties and teach composting techniques, 2) work with local landscapers to demonstrate environmentally friendly landscaping, and 3) provide direct technical assistance to residents. Local grade schools and other educational institutions have been contacted and teams have been established to develop curriculum items on urban non-point source pollution issues such as wetland and stormwater management, pollution reduction, and ecosystem diversity. The Tollgate wetlands will become a living classroom for field trips for students of all ages.

A reptile and amphibian roundup will take place this spring. Specimens of varying reptiles and amphibians will be caught, tagged, and released in the Tollgate wetlands by second and fifth grade classes from a nearby grade school. This exercise is but one example of programs designed to involve parents and children in understanding the complexities and needs of urban wetlands, and the creatures that live therein.

Other local environmental groups are participating by building birdhouses and educational kiosks along the pathway adjacent to the wetlands, and providing information to assist in educating wetland users from the broader community. Service clubs will be invited for box lunch tours. This broader outreach will have an impact on the attitudes toward the overall sewer separation project in the City of Lansing.

Conclusion

Planning for this project included a public outreach and education plan. The savings to the overall project's initial and long-term costs were substantial. We developed a well-informed public who are willing to take ownership and who understand the need to change behaviors. Retrofitting established urban environments will be more successful when accompanied by education and public outreach. The ultimate success or failure of most projects depends on the willingness of the residents to change their long-standing practices pertaining to wetlands and runoff.

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Minneapolis Chain of Lakes Phosphorus Reduction Strategy

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Abstract

The Minneapolis Chain of Lakes is a group of five heavily used urban lakes located approximately 1.5 miles southwest of downtown Minneapolis, MN. The 8,000-acre watershed was developed over the last 100 years and is now completely urbanized. Watershed development and human activities have lead to water quality degradation over the last 40 years. In 1991, a diagnostic study of urban runoff and lake water quality lead to the development of an 8-year, multi-million dollar watershed management effort. A key component of the management strategy was quantification of phosphorus load reduction goals. The watershed management plan is based on reductions of inflow phosphorus loads to the lakes to attain long-term water quality goals. The phosphorus load reduction strategies were developed for each of the lakes based on lake water quality modeling and watershed load analysis. The load reduction goals established for each of the lakes are (percent of total watershed load): Brownie - 10%; Cedar - 40%; Lake of the Isles - 20%; Calhoun - 30%; and Harriet - 20%. The two main structural best management practices (BMPs) being implemented are wetland/pond systems and grit chambers. The structural BMPs, in conjunction with increased street sweeping and public education, are being implemented to reduce phosphorus loading from this fully developed watershed.

The Minneapolis Chain of Lakes are located 3 miles southwest of downtown Minneapolis, MN. The Chain's five lakes are the central natural resource feature of the Minneapolis Chain of Lakes Regional Park. The regional park receives over 2.25 million visitors per year, making it the second most heavily used regional park in the Twin Cities Metropolitan Area. The chain receives urban runoff from a fully developed 8,000-acre watershed that includes portions of Minneapolis and the adjoining suburban communities of St. Louis Park and Edina. Major land use categories presently include residential development (51%), industrial/commercial (19%), and open space (14%). The lakes in the Chain (Brownie, Cedar, Isles, Calhoun and Harriet) are interconnected with navigable channels, culverts or pumping systems. The lakes and the watershed drain from north to south, flowing from the upper chain (Brownie to Calhoun), where water is pumped to Harriet, which discharges to Minnehaha Creek, a tributary to the Mississippi River. Demand for this study and watershed management effort arose from public concerns over increased recreational use of the lakes and attendant deterioration in water quality.

Diagnostic Study

The first step in designing the watershed diagnostic program was an assessment of previous watershed runoff studies. Sixteen subwatersheds were selected for stormwater runoff monitoring in 1991. The selected subwatersheds accounted for 86.8% of the runoff from the watershed. Seventy-three percent of the residential area and 21% of the commercial area in the watershed were included in the monitored areas. Two subwatersheds, the Bass and Twin Lake drainage areas, drain through wetland complexes prior to discharge to the lakes. These subwatersheds were combined with other sample sites for continuous runoff monitoring.

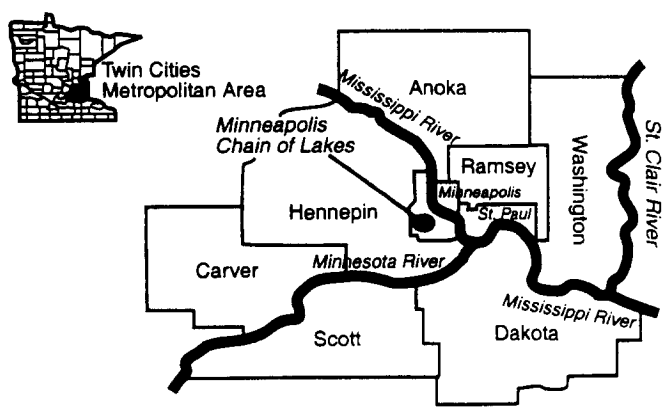

Figure 1. Chain of Lakes watershed location map (after Osgood, 1998)

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Minneapolis Chain of Lakes watershed concentration ranges for nutrients were similar to those of the Nationwide Urban Runoff Program (NURP) (Oberts, 1983). Export rates developed from this study were typically below those found in other studies (Wilson and Brezonik, 1998). The stormwater runoff characteristics of importance are listed below:

- The range of pollutant concentrations were consistent with concentrations found in the NURP studies.
- The mass of phosphorus and nitrogen input into the Chain of Lakes from stormwater in 1991 was high enough to cause water quality degradation.
- Most of the measured pollutants exhibit some type of seasonal trend, with most concentrations highest in the spring of the year following snow melt.
- Most of the measured pollutants do not show any major trends among watersheds.
- The heavy metal concentrations in stormwater, while at times quite high, rarely exceed state water quality standards. Relatively low heavy metal concentrations reflect the primarily residential nature of the Chain of Lakes watershed (Wilson, 1993).

Data collected during a 1991 diagnostic study indicated that the lakes exhibit physical and chemical processes indicative of moderately eutrophic lakes, including hypolimnetic oxygen depletion, anoxic sediment phosphorus release, and late-summer decline in water transparency. Based on a variety of trophic indicators, Jensen and Brezonik (1998) concluded that Lake Calhoun, Lake of the Isles, and Cedar Lake are eutrophic and that Lake Harriet is mesotrophic. The water quality studies also examined historical water quality data and the results of paleolimnological studies; this data indicated that water quality in the lakes has significantly declined since the early 1900s, but has been relatively constant since the early 1970s. The lakes seem to have attained a degree of equilibria with watershed nutrient inputs (Brugam and Speziale, 1983; Lee, 1993).

Vollenweider (1976) examined lake eutrophication with regard to areal based watershed loadings of phosphorus and the impact upon the trophic status of a lake with regard to lake mean depth, surface area (and, as such, volume) and flushing rates. Loading levels which exceed Vollenweider's permissible level would lead to increases in lake productivity and accelerated eutrophication. Based on the 1991 loading levels, Lake Harriet's loadings are only slightly above the permissible level. Loadings to Lake of the Isles exceed the permissible level by a factor of 9.3, Cedar Lake exceeds permissible levels by a factor of 5.1, and Lake Calhoun exceeds the permissible level by a factor of 3.8. Lake of the Isles had the highest average phosphorus concentration over the 1991 - 1996 growing season, followed by Lake Calhoun, Cedar Lake, and Lake Harriet (Jontz and Lee, 1998).

Watershed Management Plan Development

Phosphorus and, to a lesser extent, nitrogen, are important in the eutrophication process in lakes. Excessive inputs of phosphorus to lakes will lead to an increase in fertility and subsequent algal blooms. Findings of the 1991 Storm Water Runoff study reinforced these concerns and were useful in designing a watershed management plan. Barr Engineering Company (1992) and Lee (1993) used computer models to calculate the pollutant concentrations associated with particular land uses within the Chain of Lakes watershed. Those concentrations became the basis for determining the estimated total annual mass of pollutants (loadings) contributed to the lakes.

Loading Rates (kilogram/hectare of land/year)

	Total Phosphorus	Total Nitrogen
Open/Green Space Land Use	0.13	1.59
Residential Land Use	0.54	2.52
Comm/Mixed Use	0.41	2.61

Storm event data was reduced using FLUX (Walker 1986), an interactive computer program that calculates pollutant load and flow weighted mean concentrations (FWMC). FLUX was used with the continuous flow records and parameter concentrations to develop a FWMC and loading (in kg/yr) for sites where both flow and sample analysis data were available. Annual event mean concentration and literature values were also used to refine FWMC and load for the monitored and unmonitored watersheds (Oberts, 1983; Oberts, 1990; Bannerman, et al., 1983).

In 1991, stormwater contributed 50% of the Lake Harriet phosphorus budget, and the pumped discharge from Lake Calhoun provided another 21%. Stormwater loadings accounted for over 80% of the phosphorus input to the upper Chain of Lakes. The total mass of phosphorus and nitrogen input to the Chain of Lakes from stormwater in 1991 was much lower than would be expected based on published nutrient yields from other urban watersheds (Mulcahy, 1989).

Table 1. Observed and Modeled Lake Water Quality Conditions for 1991-1995 and Watershed Phosphorus Loads for 1991 from BATHTUB

Five-year Mean Annual Conditions

Lake	Cedar	Isles	Calhoun
Load Source (kg phosphorus/yr)			
Precipitation	38.5	23.4	95.5
External load	220.3	167.8	536.1
Internal load	76	34.3	465
Other[1]	9.6	66.5	112.3
Total (kg/yr)	344.4	292	1208.9
In-Lake Water Quality[2]			
Observed Total P (µg/L)	49	59	40

[1] Advective/diffuse inflows
[2] Observed based upon 1991-1995 average lake conditions and 1991 monitored loadings

continued

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Table 1. Continued

Predicted Water Quality (BATHTUB model outputs)

Lake	Cedar	Isles	Calhoun
Load Source (kg phosphorus/yr)			
Precipitation	38.5	23.4	95.5
External load	220.3	167.8	536.1
Internal load	76	34.3	465
Other[1]	9.6	66.5	112.3
Total (kg/yr)	344.4	292	1208.9
In-Lake Water Quality[3]			
Predicted Total P (µg/L)	33.8	51.1	33.2

[3] Predicted water quality based upon BATHTUB model outputs. Observed TP concentrations greater than predicted concentrations indicates internal phosphorus loading.

Water budget and nutrient concentrations from the storm water sampling were input to the BATHTUB lake model to determine lake interactions. BATHTUB is a lake model designed to facilitate the application of eutrophication models to lakes and reservoirs (Walker 1986). BATHTUB does nutrient and water balance calculations, and predicts water quality conditions using empirical relationships developed by Walker (1985). The model also provides a breakdown, by subwatershed, of nutrient loads to each lake. The BATHTUB model provides diagnostic variables that summarize the physical, chemical, and biological interactions occurring in the lake. These assessments were used to evaluate the management options for each of the lakes.

A critical part of the management plan was the development of lake water quality goals and watershed loading reduction strategies by a Citizens Advisory Committee (Lee, 1995; Derby and Lee, 1998). The long-term goals recommended by the committee are based on an analysis of historical, existing and predicted lake conditions. The goals represent water quality conditions that are close to predicted historical levels and are attainable by reducing watershed phosphorous loads (Heiskary and Walker, 1988; Wilson and Walker, 1989). The short-term goals represent immediate lake improvements, attainable through watershed management and inlake manipulations, while the long-term goals require the lakes to equilibrate with reduced watershed loads.

The long-term (5-10 year) water quality goals, historic water quality, and 1992 average conditions for each lake are:

Lake	Predicted Historical Total Phosphorous*	1992 Summer Mean Total Phosphorous	Long-term Mean TP Water Quality Goal
Brownie**	24	34	35
Cedar	21	48	25
Lake of the Isles	28	74	40
Calhoun	18	54	25
Harriet	19	44	20

* Predicted historical phosphorus derived from Vighi and Chiaudani (1985) which uses lake water chemistry, specifically alkalinity, to predict unimpacted water quality.

** Brownie Lake conditions based upon 1991 data.

In the Chain of Lakes, as in many lake systems, phosphorus is the limiting nutrient. Reducing watershed loadings of phosphorus and interrupting internal phosphorus recycling will reduce blue-green algae numbers and decrease the frequency of nuisance algal blooms. Thus, the watershed management plan is based on reducing phosphorus loads. Phosphorus load reduction strategies were developed for each of the lakes, based on lake water quality modeling and watershed load analysis. The load reduction goals (as percent of total watershed load) established for each of the lakes are as follows:

Lake	Phosphorus Load Reduction
Brownie	10%
Cedar	40%
Lake of the Isles	20%
Calhoun	30%
Harriet	20%

Implementation of Best Management Practices

Priority management areas were chosen based on the results of the 1991 Storm Water Monitoring Program and subsequent modeling efforts. Due to the primary concern for lake eutrophication problems, phosphorus loads were used to determine priority watersheds. The priority watershed management areas were ranked, based upon annual phosphorus load.

The best management practice selection process included a series of analyses that factored in a range of watershed and BMP characteristics. The pollutant of concern (phosphorus) dictated the main emphasis of the selection process. In many cases, the watershed characteristics of each subwatershed area, as well as site availability and characteristics of each site, limited the size and type of BMP. For larger pond/wetland systems, the BMP space requirements were such that large areas of open space were needed; in all cases, these were parklands. Consideration for adjoining neighborhoods and environments/parks required that all BMPs be designed to aesthetically blend in and become a neighborhood asset rather than a "mosquito infested swamp," as many opponents feared. The cost of BMP installation, and amortized costs on the basis of pollutant removal efficiency over the life of the project, were also factored into each BMP decision. Finally, the maintenance effort for each BMP over the long term dictated which agency would be responsible for long-term management.

As a result of the BMP selection process, the following watershed best management practices were selected as part of the watershed management plan:

Non-structural BMPs

- Water Quality Education
- Catch Basin Stenciling
- Lawn Care Education

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- Street Sweeping/Cleaning
- Goose Population Management
- Erosion Control Ordinance

Structural BMPs

- Grit Chamber Construction
- Stormwater Wetland/Detention Basin Construction

The Minneapolis Chain of Lakes Clean Water Partnership Implementation Project began in 1994. Implementation of watershed management practices was through the cooperative efforts of the Minneapolis Park & Recreation Board, the cities of Minneapolis and St. Louis Park, the Minnehaha Creek Watershed District, and the Minnesota Pollution control Agency. As first steps, regenerative air street sweepers were purchased and a watershed-wide education program was implemented. The Minneapolis City Council enacted and implemented a construction erosion control ordinance for all construction activities in 1996. Construction of structural BMPs in the Chain of Lakes watershed began in 1995. Total projected project costs for the seven year program are detailed in Table 2.

In-lake management techniques were also implemented as part of the comprehensive management effort. Prior to finalizing the management plan, the in-lake management alternatives were subjected to the analysis suggested by Cooke, et al., (1993). As part of the comprehensive management plan, whole-lake alum treatments have been completed (Cedar and Isles) or are scheduled (Calhoun) for the lakes. The plan also includes annual aquatic macrophyte harvesting.

Structural Best Management Practices

Priority subwatersheds were targeted for BMP installation. The structural BMPs, in conjunction with increased street sweeping and public education, are being implemented to reduce phosphorus loading. Grit chambers and constructed wetland/detention basins will be responsible for the largest reductions in watershed phosphorus loads to the lakes.

Table 2. Minneapolis Chain of Lakes - Clean Water Partnership Implementation Project Costs for 1994 - 2001 (after Panzer, 1998)

Expenditures		Cost
Education		$374,500
Watershed Management Practices		
Grit chambers	757,580	
Street cleaning	926,500	
Stormwater ponds & wetlands	4,594,500	
Other	7,500	$6,286,080
In-Lake Improvements		
Alum	237,950	
Erosion protection	284,890	$522,840
Monitoring Programs		
Lakes	465,000	
Storm water	433,740	
Beach	37,490	$36,230
Total Expenditures		$8,119,650

Grit Chambers installed within the existing storm sewers settle sediment from storm water. Grit chambers have been or will be constructed in the storm water drainage systems of priority subwatersheds that do not have sufficient open space for detention basins or wetlands. The efficiency of sedimentation (and corresponding pollutant) removal is directly related to the size of the structure — the larger the structure the greater the pollutant removal. Stormwater runoff was modeled using the P-8 model to predict the pollutant removal by grit chambers for flows expected during a typical wet year, dry year, and normal year (HDR Engineering, 1992). The results of this modeling were:

Total Suspended Solids	3% to 58% removal
Total Phosphorus	1% to 26% removal
Lead	2% to 56% removal
COD	1% to 35% removal

The range in pollutant removal reflects the range of sizes of grit chambers modeled (10 different sizes). The highest removals were achieved for a small subwatershed which was 100% impervious. The systems are designed for a 10-year storm (2.29 inches/hour) with a target reduction of 33 - 42% for total suspended solids (Asgain, 1994).

The greatest pollutant loading occurs during late winter snow melt and early spring rainfall. Therefore, it is recommended that these structures be cleaned two times per year: (1) in the fall or winter, in order to have maximum capacity for settling solids in the snowmelt and (2) in the spring, in order to clean solids accumulated from the snowmelt and to maximize capacity for the remainder of the year.

The major BMP expenditures for the Chain of Lakes watershed management effort have been constructed wetlands/detention basins. The priority subwatershed analysis results showed that most of the phosphorus entering Cedar Lake came from the subwatershed draining through Twin Lakes. Twin Lakes (a shallow wetland) receives runoff from about 1,600 acres of residential and commercial land, discharging over half of Cedar Lake's water and over 60% of the external phosphorus load. The area treated by this project represents about 85% of Cedar Lake's contributing subwatershed.

A 1994 feasibility study evaluated several BMP alternatives and selected a combination of measures to address non-point source pollution:

1. Construction of a wet detention pond/wetland in Twin Lakes Park to treat runoff from approximately 1,600 acres before entering Twin Lakes.
2. Excavation of the Twin Lakes basin to improve the lake's aesthetic quality and increase phosphorus removal effectiveness (before discharge to Cedar Lake).

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3. Construction of a wet detention pond and shallow wetland on property owned by Minneapolis Park & Recreation Board, immediately upstream from Cedar Lake. This would further treat the discharge from Twin Lakes and the direct runoff diverted from Cedar Lake (Panzer, 1998).

Modeling results for the Cedar Meadows/Twin Lakes watershed detention project indicated that the system would remove 43% of the Cedar Lake watershed phosphorus load. Work on the Cedar Lake watershed project was completed in the spring of 1996.

A similar wet detention pond system is proposed for Lake Calhoun in 1998. The project is currently in the final design phase. The Lake Calhoun detention system will treat runoff from an 897-acre subwatershed draining from the southwest. This subwatershed currently contributes 37% of the total phosphorus load to Lake Calhoun. Modeling for the southwest Lake Calhoun subwatershed detention system indicates that the system would remove 48% of the subwatershed phosphorus load and 13% of total Lake Calhoun watershed phosphorus load.

Conclusions

The use of a wide array of tools made possible the development of realistic and attainable water quality goals for the Chain of Lakes. The realities of urban limnology, that is, the magnitude of stresses placed on the aquatic resources, require that new, alternative BMP strategies be developed. In fully developed watersheds such as the Minneapolis Chain of Lakes, retrofit opportunities have necessitated the conversion of open space (usually parkland) to detention systems. This conversion requires that aesthetic and neighborhood considerations play a major role in the design process.

The practice of urban limnology requires the adaptation of ecological concepts and modeling tools to describe and predict stresses placed on urban lakes. Monitoring and modeling efforts prior to implementing the watershed management plan informed goal setting and allowed for prediction of water quality improvements due to management practices. It is widely understood that BMPs alone will not prevent lake water quality degradation due to watershed urbanization. In the case of the Chain of Lakes, historical data showed that most of the stresses occurred during watershed development. The lakes have reached a new equilibria with current watershed loadings, such that reductions in external and internal phosphorus loads can be expected to cause measurable improvements in water quality. Thus, while BMPs cannot be expected to prevent declines, implementation of watershed practices after lakes have reached this new equilibria can be expected to lead to improvements when implemented in conjunction with in-lake management.

The benefit derived from attaining these goals is improved lake trophic status, which will lead to a reduction in algae blooms and aesthetically improved lake conditions. Attainment of the goals will shift Lakes Calhoun and Cedar from their present eutrophic conditions to slightly mesotrophic states, while keeping Lake Harriet mesotrophic. Lake Calhoun and Cedar Lake present the best opportunities for water quality improvement through implementation of watershed management practices. Monitoring results suggest that shifts in water quality are already occuring (see Figure 2).

Lake monitoring results for 1996-97 show that Cedar Lake and Lake of the Isles water quality is now approaching the lakes' goals (see Table 3). The 1993-97 period included years of higher than average precipitation and some of the water quality improvements noted may be due to increased lake flushing. Monitoring over the next 3 years will further indicate the stability of water quality changes due to watershed management.

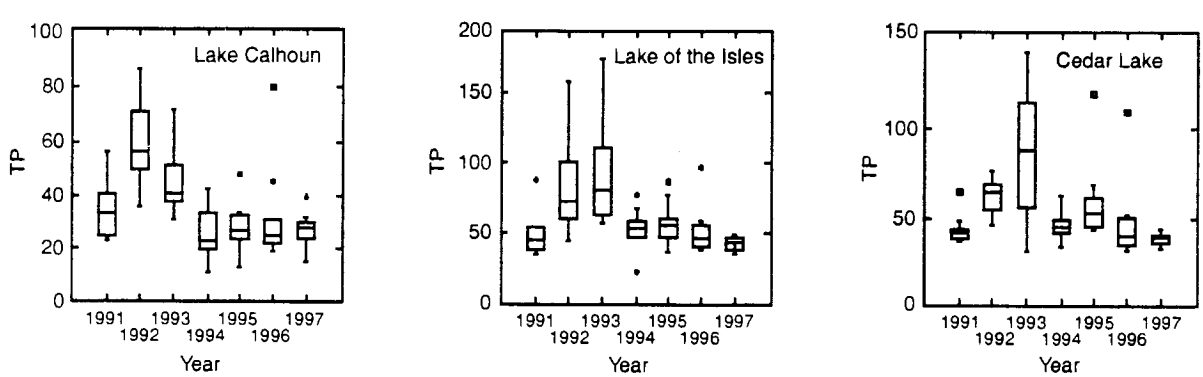

Figure 2. Growing season total phosphorus data for 1991 - 1997 for the Minneapolis Chain of Lakes (values in µg/L; mean, 25th and 75th percentile box plot).

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Table 3. Post-implementation Watershed Load Projections, Predicted In-lake Total Phosphorus Concentrations, and Mean Growing Season Total Phosphorus Concentrations (1996-1997)

Lake		Cedar	Isles	Calhoun
Load Source (kg phosphorus/yr)				
Precipitation		38.5	23.4	95.5
External load		154.4	155.2	366.7
Internal load[5]		38	17.15	232.5
Other		9.2	54.4	107.3
Total (kg/yr)		240.1	250.15	802
Loading reduction (kg phosphorus/yr)		104.3	41.85	406.9
In-lake Water Quality				
Predicted	Total P (µg/L)[6]	30	42	22
Lake goals	TotalP (µg/L)	25	40	25
G.S. Mean (96-97)	Total P (µg/L)	27.5	37	27.5

[5] Internal load reduced by 50% (representing average reduction over life of treatment).

[6] Predicted values derived using Dillon and Rigler (1974).

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Restoration in the Sunshine: Retrofitting the Watersheds of Two Urban Lakes in Florida

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Because of its karst geology and generally flat terrain, Florida is dotted with numerous shallow lakes. The shallowness of the lakes, combined with generally warm temperatures and long periods of sunlight, make these lakes particularly susceptible to anthropogenically accelerated eutrophication, resulting in poor habitat and water quality. In response to citizen complaints regarding degraded water quality of two lakes in Pinellas County, the Southwest Florida Water Management District and two local governments conducted diagnostic/feasibility studies to develop restoration strategies and projects. Although the technical aspects of retrofitting the watersheds and restoring the lakes were daunting, the inter-governmental coordination, regulatory, and social hurdles were far more difficult to overcome and have caused significant delays to restoration activities. This paper will review the watershed retrofit and restoration activities for Lakes Maggiore and Seminole, and highlight several unique technical solutions proposed for the projects. The many pitfalls encountered while conducting these two projects, and strategies for avoiding them, will be discussed.

Introduction

Florida's landscape is dotted with over 7,000 generally shallow lakes. Many of these lakes are found within urban areas or areas undergoing rapid urbanization, particularly in the Central Florida corridor which includes the metropolitan areas of Tampa/St. Petersburg, Lakeland/Winter Haven, and Orlando. Because of Florida's subtropical location and climate, the growing season for algae and other aquatic plants is typically year round. Adding pollutant and nutrient-laden stormwater runoff to these shallow urban lakes creates the potential for accelerated eutrophication and accompanying problems of algal blooms, fish kills, cattail and hydrilla infestations, and the general degradation of the ecological and aesthetic qualities of these systems. Although Florida is a leader in stormwater regulation and treatment, state stormwater regulations have been in place only since 1984, and most of the urban development that is responsible for degrading these urban lakes occurred long before.

Techniques for effectively treating stormwater (e.g., wet detention) generally are well established, but depend on a minimum amount of open land. The difficulties of retrofitting stormwater systems in urban areas are thus largely a matter of having undeveloped land available upon which to construct treatment systems. Obviously, undeveloped land is infrequently available, and if it is, it is typically quite expensive. Although the technical difficulties of retrofitting urban watersheds can be daunting, the political and regulatory hurdles can be even more challenging. Overlapping municipal and agency jurisdictions and responsibilities frequently result in "turf wars" that can slow restoration projects and ultimately increase the cost of implementation. Administrative requirements within local governments and agencies to develop agreements for funding and project contracts can also significantly delay and increase overall retrofit costs.

In this paper, case studies of two urban lake restoration projects currently underway in Pinellas County, FL, will be presented. These two projects demonstrate the great opportunities, as well as the prodigious challenges encountered while attempting to retrofit highly developed urban watersheds and improve the ecological and aesthetic qualities of important urban resources.

Lake and Watershed Descriptions

Lakes Maggiore and Seminole are located in Pinellas County, FL (Figure 1). Lake Maggiore is a 386-acre lake located just south of downtown St. Petersburg, while 684-acre Lake Seminole is near the cities of Largo and Seminole in central Pinellas County (Figure 1). Land use in the watersheds of both lakes (Table 1) is largely residential, light industrial, and commercial, although there are public parks located on the shores of both lakes. Additionally, there is a large upland habitat preserve (Boyd Hill Nature Park) on the western shore of Lake Maggiore. Of the developed portions of the watersheds of both lakes, only a small fraction (approximately 11%) receives stormwater treatment based on 1984 state regulations.

Both lakes support boating, fishing, and water skiing, although the degraded conditions have curtailed these activities, particularly in Lake Maggiore. Lake Maggiore was the site of a major annual hydroplane race until a driver

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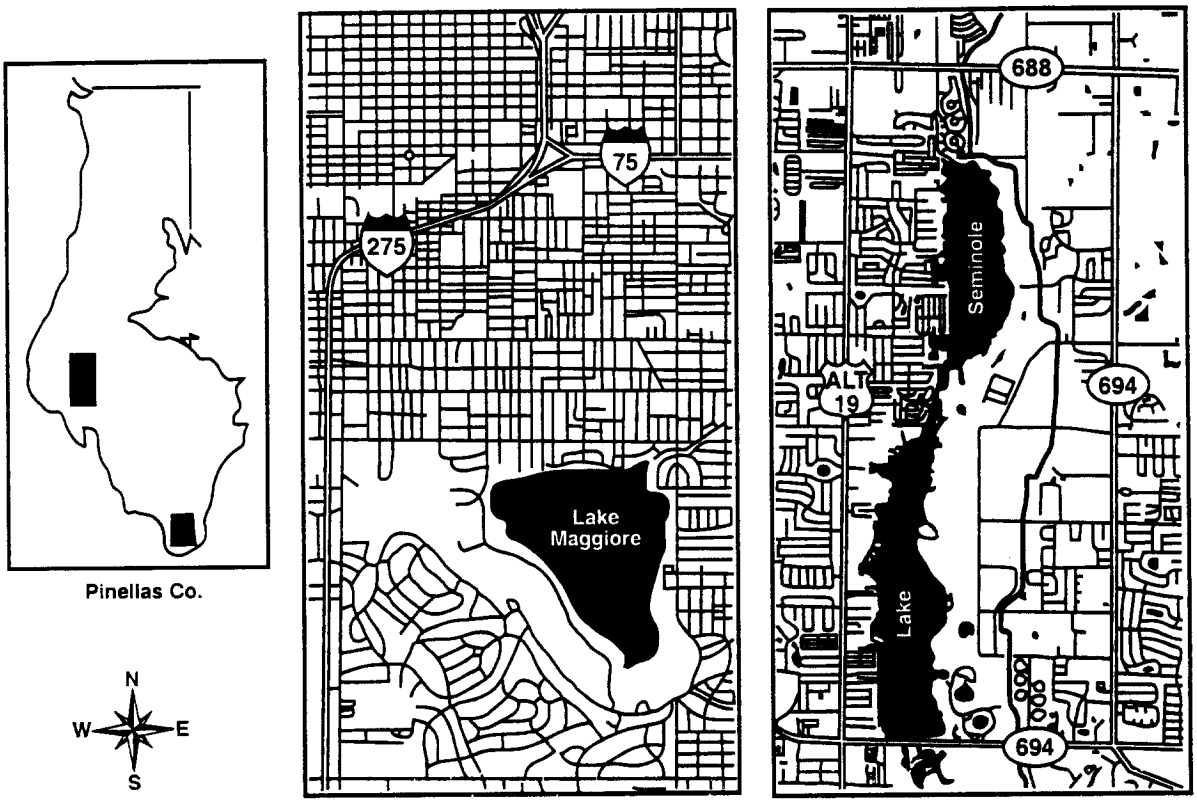

Figure 1. Lake Maggiore and Lake Seminole.

Table 1. Land Use Data for Lake Maggiore and Lake Seminole Watersheds, Pinellas County, Florida.

	Lake Maggiore[1]		Lake Seminole[2]	
	Acres	Percentage	Acres	Percentage
Commercial/Industrial/Public	94	4.1%	510	15%
Pond	64	2.8%	N/A	N/A
Undeveloped/Open/Recreational	*1448*	*63.2%*	*2554*	*73%*
Low Density	671	29.3%	1766	50%
Medium Density	690	30.1%	521	15%
High Density	87	3.8%	267	8%
Overall Total	*2291*	*100%*	*3489*	*100%*

[1] Source - Harper, H. and J. Herr, 1994.
[2] Source - Southwest Florida Water Management District and Pinellas County Department of Environmental Management, 1992.

was thrown from his boat during a crash, and subsequently developed a serious waterborne infection which resulted in hospitalization. Since then, conditions on the lake have been deemed by the City of St. Petersburg to be too dangerous for water contact sports.

Diagnostic/feasibility studies, conducted for both lakes in the early 1990's, identified the source and magnitude of nutrients and pollutants entering the lakes. Not surprisingly, both lakes have high levels of nitrogen, phosphorus, and chlorophyll (Table 2) and are highly productive. Florida Trophic State Indices (TSI) were calculated for each lake and annual averages ranged between 75 and 85 for both, indicating eutrophic to hypereutrophic conditions. Comparisons of water quality with other Florida lakes (Table 3) indicate that both lakes are among the most degraded in the state. Biological communities of both lakes were dominated by undesirable species of fish, algae, macrophytes (especially cattails), and invertebrates.

Large monocultures of cattails in both lakes have contributed to general habitat degradation and significant addi-

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Table 2. Water Quality Data for Lake Maggiore and Lake Seminole, Pinnellas County, FL.

Parameter	Lake Maggiore[1] 1992-1997 Median	Range	Lake Seminole[2] 1991-1997 Median	Range
Total Phosphorus (mg/L)	0.059	0.005-0.394	0.10	0.02-1.01
Total Nitrogen (mg/L)	2.42	0.437-9.04	1.90	0.02-7.53
Chlorophyll *a* (μg/L)	87.8	14.8-620	62.6	5.34-144
Secchi Depth (m)	0.28	0.11-1.1	0.38	0.15-0.9
Florida Trophic State Index	82.7	55.4-104	80.8	36.4-111.4

[1] Source - Southwest Florida Water Management District, unpublished data.
[2] Source - Pinellas County Department of Environmental Management, unpublished data.

Table 3. Comparison of Water Quality Data for Lake Maggiore and Lake Seminole (Pinellas County, FL) to Water Quality Data for Florida Lakes Reported by FDEP (Friedemann and Hand, 1989). The Florida Percentile Shows the Ranking for Each Parameter among the Percentile Distribution Listed by FDEP for Median Values for their Florida Lake Database for 1970 to 1987.

Lake Maggiore[1]	Median Value 1992-1997	Percentile of FL Median Values
Total Phosphorus (mg/L)	0.059	45 (864 lakes)
Total Nitrogen (mg/L)	2.42	86 (781 lakes)
Chlorophyll *a* (μg/L)	87.8	94 (550 lakes)
Secchi Depth (m)	0.28	3 (782 lakes)
Florida Trophic State Index	82.7	97 (756 lakes)

Lake Seminole[2]	Median Value 1992-1997	Percentile of FL Median Values
Total Phosphorus (mg/L)	0.10	67 (864 lakes)
Total Nitrogen (mg/L)	1.90	80 (781 lakes)
Chlorophyll *a* (μg/L)	62.6	88 (550 lakes)
Secchi Depth (m)	0.38	5 (782 lakes)
Florida Trophic State Index	80.8	96 (756 lakes)

[1] Source - Southwest Florida Water Management District, unpublished data.
[2] Source - Pinellas County Department of Environmental Management, unpublished data.

tion of organic materials to the sediments. Additionally, the profusion of tall stands of cattails have generated numerous citizen complaints regarding obstructed views of the lakes (particularly Lake Maggiore) and the role cattails play as havens for mosquitoes, rodents, and undesirable reptiles (alligators and snakes).

Recommendations from both diagnostic studies centered on treating stormwater runoff, coupled with specific in-lake restoration projects, to substantially improve water quality in the two lakes. For Lake Maggiore, this required whole lake dredging and retrofitting the watershed to appreciably reduce nutrient sources to the lake. For Lake Seminole, it is expected that some dredging, along with retrofitting, will be required.

Restoration Approach - Lake Maggiore

To initiate the Lake Maggiore restoration project, the city approached the district to fund a diagnostic/feasibility study. The district agreed to fund a $160,000 study with in-kind contributions from the city. The city's Engineering Department was designated to manage the entire lake restoration project. Their first task was to establish an advisory committee charged with providing input to, and overseeing the progress of, the diagnostic study. The committee, which included local citizens, representatives from environmental organizations, and government agencies reviewed the results of the diagnostic study and selected a TSI of 60 as the target water quality goal for the lake.

To achieve the target TSI for Lake Maggiore, it was determined that retrofitting as much of the watershed as possible was required. This was obviously a technically challenging and expensive proposal because of the lack of available undeveloped land for construction of standard wet detention systems. Although there is substantial undeveloped city-owned land surrounding the lake, including a public park and the Boyd Hill Nature Preserve, a strict city ordinance prohibits stormwater treatment facilities on these lands. Thus, a creative and less land-intensive solution was required, and the ultimate treatment option chosen was alum injection.

Alum (aluminum sulfate) has long been used in drinking water treatment and wastewater treatment to remove

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particulate matter and phosphorus (P). In Florida, the use of alum to remove P from stormwater runoff entering lakes has been effective as a restoration tool for numerous urban lakes, particularly in the Orlando area. Treatment typically consists of injecting a calibrated dose of alum solution into a stormwater stream to precipitate the dissolved and particulate P and thereby reduce the in-lake P concentration; and ultimately reduce algal populations within the lake. Alum injection systems are compact, and this was viewed as a desirable feature for installation at Lake Maggiore. All the pumps, control panels, piping, and alum storage tank could be placed in a very small area, approximately the size of a one- or two-car garage. Additionally, a number of stormwater outfalls to the lake could be treated from a single system through a manifold. Finally, the cost of constructing alum injection systems was considerably less than condemning and purchasing property for construction of standard wet detention systems.

Five alum injection sites were constructed to treat over 63% of the runoff from the watershed and reduce the amount of P entering the lake by 80%. Construction was completed in the fall of 1997, and the injection devices should be in operation by February of 1998. Stormwater from the remaining untreated portion of the watershed (eastern side of the lake) will be directed through a wet detention system to be constructed as part of the proposed dredging project.

Whole lake dredging was proposed as another major restoration project for Lake Maggiore. The lake bottom is covered with over 2.3 million cubic yards of organic deposits and fine sediments which are a source of nutrients and create significant oxygen demand during the warmer months. Thus, to reach the TSI goal set for the lake it would be necessary to remove the bulk of these sediments in addition to treating the stormwater. The land requirements needed for this project were particularly difficult to meet since it was not possible to use the city-owned land around the lake for processing or disposal of the dredge spoil. The cost of transporting the spoil to a landfill or other offsite property was prohibitive, so a unique solution was proposed to dewater the sediments in an effort to reduce spoil disposal costs.

The proposed solution for dewatering the sediments, developed for the phosphate industry to settle out clay particles, involved introducing organic polymers into the dredge spoil, followed by screening the material to separate out the aqueous fraction. Untreated dredged material from Lake Maggiore is expected to have a solids content of 4-6%, while the polymerized material is expected to have a solids content of 10-15% immediately after treatment and 30-40% after rapid drying. This treated spoil material could be easily transported within seven days of dredging (weather permitting).

Despite reducing the amount of spoil to be transported, it still would not be possible to cost- effectively transport all the sediments to a landfill. Therefore, an option was investigated that involved pumping a portion of the dredge spoil to several old borrow pits located within the city. Because of opposition from the public and the State Departments of Transportation and Environmental Protection, this option was abandoned. Subsequently, another option was developed that proposed that a portion of the lake could be filled with the dewatered spoil to create a public park, and a wet detention system could be built to treat the stormwater runoff from the eastern watershed. Although the proposal included filling in 34 acres of the 386 acre lake to accomplish this task, it was believed that the water quality and habitat benefits were worth transforming that amount of lake bottom. However, regulatory and permitting agencies did not completely agree with our cost/benefit analyses and at this time the permit is still being reviewed by the US Army Corps of Engineers. When, and if, the project is approved, it should take 18 months to two years to complete.

Several smaller projects were proposed and have been implemented for Lake Maggiore, including the construction of a new operable outfall structure from the lake to allow future drawdowns and sediment consolidation, the removal of large cattail stands on the eastern shoreline and revegetation of the site with desirable aquatic plants, and the purchase of an aquatic plant harvester to remove cattails and other undesirable aquatic plants on an ongoing basis.

Restoration Approach - Lake Seminole

As with Lake Maggiore, a local government, Pinellas County, sought funding from the District for a diagnostic/feasibility study of Lake Seminole. The District expended over $400,000 on a detailed study to document the sources and magnitude of nutrients and pollutants entering the lake. Pinellas County put its Department of Environmental Management (DEM) in charge of the project. Like the city, DEM formed an advisory committee comprising similar interest groups and agencies. Once the study was completed, the committee recommended a water quality target TSI of 60, the same as for Lake Maggiore.

One of the conclusions of the diagnostic study, that the eutrophic condition of the lake was the result of untreated stormwater runoff entering the lake, was similar to that of the Lake Maggiore study. In contrast to Lake Maggiore, the Lake Seminole study identified only small pockets of organic sediment within the lake basin and, thus, no large dredging project was proposed. Retrofitting the Lake Seminole watershed posed the same difficulties encountered at Lake Maggiore, since there was only a small parcel of open land potentially available for standard wet detention treatment systems. The parcel, which the county purchased for $1.9 million, comprised five usable upland acres on the shore of the lake and was the last open land available around the lake.

The county has not yet settled on one type of retrofit system for the watershed. Rather, a consultant is preparing a watershed management plan that is expected to pro-

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pose a mix of retrofit solutions. The original diagnostic study identified several sub-basins within the watershed that contribute significant amounts of nutrients to the lake, and within which several remedial actions could be implemented immediately. On the five-acre parcel, a wet detention system was proposed that would incorporate a state-of-the-art system employing a residence time of nearly 14 days. At another site, an existing retention pond was redesigned to improve its treatment capabilities by reducing the slope, meandering the banks of the pond, and planting native wetland vegetation. An operable structure has been installed at the pond outfall to allow for drawdowns and cattail removal. Finally, untreated runoff from a 15-acre residential area will be routed through a refurbished wetland on the property of a local junior college. Like the five acre parcel, this system will provide a 14-day residence time prior to discharge. Once the watershed management plan is complete, additional retrofit projects will be proposed and implemented.

A unique retrofit project currently being considered for the restoration of Lake Seminole incorporates two state-of the-art technologies. This project proposes to link a vortex trash and sediment collector (CDS Technology) with an alum injection system. The vortex system would remove large sediment particles and trash while the alum injection system would inactivate the soluble phosphorus in the runoff. Both systems require only small amounts of land for installation and may fit into existing stormwater collection systems. Such integrated systems may be especially well suited to the Lake Seminole watershed, since many of the streets on the western (heavily developed portion) shore of the lake collect stormwater and discharge directly to the lake through pipes at the end of each street. A total of 75 such discharges have been identified along the western shoreline of Lake Seminole.

Like Lake Maggiore, Lake Seminole has expansive growths of cattails that have degraded the littoral habitat and caused citizen complaints about obstructed lake views. The county experimented with several cattail removal/ revegetation projects and, as with the city, was provided with an aquatic plant harvester purchased by the district. These projects met with some difficulty. Particularly the revegetation portion, where grass carp, introduced earlier to eradicate hydrilla, ate the newly planted vegetation. Ultimately plastic fences were constructed to bar the carp from these sites until the new vegetation became properly established; however, cattail removal has proven to be an expensive and problem-plagued undertaking.

Lake Seminole discharges over a fixed crest weir to Boca Ciega Bay during high water periods. A recommendation from the diagnostic study was to replace the weir with an operable structure so the lake could be periodically drawn down for sediment consolidation purposes and cattail removal. This structure is currently in the design stage and is expected to allow a maximum three feet fluctuation in lake levels.

Success is a Four Letter Word

Although funding such large scale lake restoration projects is often a difficult undertaking, all three entities involved in both projects (the district, the City of St. Petersburg, and Pinellas County) initially were enthusiastic about providing adequate funding. The district provided $5 million for each lake while the city and county were responsible for an additional $5 million each for their specific project. Ten million dollars for each project seems generous, but as the recommendations of the diagnostic studies reached the design stage it was obvious that timely implementation of the retrofit and in-lake restoration projects was going to be difficult to fully achieve. And it is well known that "time is money."

For the restoration of Lake Maggiore, the city had to win the confidence of a very active environmental group associated with the Boyd Hill Nature Preserve. Previous adversarial interactions with city government had caused this group to be suspicious of any "environmental improvement" project sponsored by the city. Although the city had disbanded the advisory committee, a number of meetings were held with this group and homeowner committees to allay concerns and, ultimately, these groups endorsed the overall project; however, they vehemently opposed any use of public lands (park land) for either retrofit or dredging. This opposition had serious financial implications for both aspects of the restoration in that (1) land would not be available to construct small ponds at the alum injection sites for collecting the precipitate prior to disposal at a landfill and (2) there would be no land available around the lake or any other nearby public land on which to place the dredge spoil. Both these difficulties required time and project redesign to solve.

The alum pond problem was resolved when the city Engineering Department approached the owners of a nearby private golf club and sought their approval to use the course's water features as alum collection ponds. In exchange for improvements to several drainage features on the golf course, club officials agreed to the city's request, and the alum injection project was able to move forward.

In contrast, the dredging was (and still is) far more challenging. In addition to the lack of available land, the dredging project ran into serious regulatory difficulties. Although it would seem the environmental benefits and public interest being served would help propel the project quickly through the permitting process, it became obvious that this was wishful thinking. In fact, because this project was not a private venture but was government sponsored, it seemed as though the restoration team was compelled to jump additional regulatory hurdles to ensure compliance with every last letter of the law. Dredging the lake presented a number of regulatory issues that were capable of being resolved relatively quickly with the state agency responsible for issuing the permit; however, because there was no spoil disposal site within the watershed, the project was redesigned to place the spoil on 34 acres of lake bottom adjacent to the eastern shore. This aspect of the project was and continues to be a permitting challenge, since placing spoil on sovereign submerged land in Florida is severely restricted and requires the approval of the Gov-

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ernor and Cabinet. Additionally, Lake Maggiore and all of the natural waterbodies of the county are considered Outstanding Florida Waters. This special protective state designation limits any developmental activities within the waterbody, especially dredging and filling.

To make this design acceptable to the permitting agencies, additional stormwater treatment was incorporated within the 34-acre spoil site, and the city designed a park with extensive boardwalks and environmental exhibits. However, the features designed into the site were of less concern to the regulatory authorities than the fill itself. Ultimately the site was redesigned to allow only eight upland acres for the park with the remaining 26 acres used for constructed wetlands and littoral zones. The state permit was obtained after over one and a half years of frustrating, contentious meetings and failed compromises, including three trips to the state capital to win approval of the Governor and Cabinet. Soon after the state permit was received and bids were being prepared for a dredging contractor, the Corps of Engineers (COE) asserted their displeasure with the design, which has further delayed permitting (in fact, the COE has still not issued a permit or ruling). The economic feasibility of conducting this project is now in question, since the various delays and project redesigns (including transporting the bulk of the spoil ten miles for disposal) have escalated the costs beyond the initial project budget.

For Lake Seminole, the regulatory issues to date have not been as onerous; however, the administrative difficulties of implementing projects have been as frustrating as the permitting problems for Lake Maggiore. Approval of funding agreements and project contracts between Pinellas County and the district was often a difficult and time-consuming ("time is money," again) effort because of the many steps involved in the approval processes for both entities. In one egregious case, a full year was spent ("time is money") developing a contract with language acceptable to lawyers and contract managers for the county and District. This issue was finally resolved when the District's project manager requested a meeting of all lawyers and contracts personnel from both entities. This illustrates the need to account for the slow pace of administrative processes in any restoration time-line.

Although the diagnostic study for Lake Seminole and recommendations for restoration activities were completed in 1992, project implementation has been slow. In addition to the contract approval process, the county has taken a very cautious approach to selecting projects for retrofitting the watershed and restoring the lake. This is the result of a proposed retrofit plan for another watershed in the county that generated extremely negative outcries from the citizens involved. Since then, county staff have developed a very deliberate process for evaluating projects for the Seminole watershed and have provided numerous opportunities for public input. Although several retrofit projects were selected for immediate design as a result of the 1992 diagnostic study, the county insisted on having a watershed management plan developed before proceeding with additional retrofit projects. In the preparation of this plan, the consultant is duplicating some of the work conducted in the original diagnostic study, and this plan will not be available until the summer of 1998.

Changes in project managers at both the district and county and changes in project implementation responsibilities at the county have resulted in miscommunication and project coordination problems that have delayed the overall restoration effort for Lake Seminole. In 1996, the county shifted responsibility for conducting the project from the DEM to the Engineering Department and simultaneously changed project managers. At about the same time, the district also changed project managers. Overall, these changes have been very positive, but have led to delays in several projects while the personnel involved became familiar with the projects and each other.

Unlike the city, the county had maintained an advisory committee of some kind since the inception of the restoration project in 1991. This committee originally consisted of staff from various government agencies, as well as concerned citizens. Initially, there was a great deal of enthusiasm from citizen members who had felt that once the diagnostic study was complete implementation of restoration projects would begin and their lake would soon be restored. Their enthusiasm was manifested in the establishment of a lake protection group that sponsored lake cleanup days, stormdrain painting efforts, and lobbying of the agencies involved to expedite restoration projects. Once the actual project timelines became clear, they became frustrated with the slow pace of implementation, their enthusiasm rapidly waned, and their attendance at the committee meetings ended. Now, the committee is composed almost entirely of agency staff and consultants and is singularly lacking in citizen participation. As with Lake Maggiore this lack of citizen involvement may hinder the implementation of future projects.

Lessons Learned (The Hard Way!)

The first step of any lake restoration project is the completion of a thorough diagnostic/feasibility study to determine the source and magnitude of pollutants influencing water quality and to develop remedial strategies. This may seem obvious, but many citizens and municipal officials see this work as a waste of money since they claim to know the cause of problems in their lake. The expenditure of millions of dollars on projects designed on anecdotal evidence would be negligent on the part of any responsible lake manager and should be vigorously resisted.

Once retrofit or in-lake restoration projects and their costs have been identified, all possible funding options should be sought. If the financing of a particular project can be split between several entities, the project is often easier to sell to those responsible for approving the funds since leveraging limited funding, especially in the form of a "public/private" partnership, is attractive to most local governments. Again, this strategy would seem obvious, but other

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state, private, and federal funding sources are often overlooked.

Solid intergovernmental cooperation and coordination are essential to the timely and effective implementation of any restoration project; however, these elements of effective communication are often lost in the "turf" battles that can occur between and inside large agencies. If permits are needed to implement a retrofit or in-lake project, ensure that all appropriate agencies and authorities (local, regional, state, and federal) are included and participate from the beginning. Costly delays to the Maggiore and Seminole projects have resulted not only from classic bureaucratic inertia, but also from not providing sufficient information to the permitting agencies responsible for approving the projects. More frequent meetings with all the regulatory personnel involved may have helped identify permitting problems, and thus lessened the time spent seeking permits and developing alternative solutions for unpermittable activities. However, the narrow interpretation by the regulating agencies of the rules governing the projects and the negligible credit received for the "net environmental benefit" of the overall restoration remains a source of extreme frustration for the project managers and citizens who will benefit from the projects.

For both Lakes Maggiore and Seminole, the most significant lesson learned was the need to thoroughly educate the public, the regulatory/permitting staff, and the politicians as to the goals, objectives, and time-lines of the overall projects. Garnering public support for lake restoration is absolutely crucial in all aspects of the project, from funding to implementation. Maintaining the public's interest in a typically lengthy restoration process requires that small, quickly implementable projects be proposed and completed to demonstrate some immediate benefit to the citizens. And, as noted earlier, adequately educating politicians and regulators will be the only way these projects can be appropriately funded and permitted in a timely manner.

Acknowledgments

The authors gratefully acknowledge the following individuals for their assistance in preparing this paper: Charles Courtney, King Engineering Associates, Inc.; Marty Kelly, Douglas Leeper, and Patricia Twardosky, Southwest Florida Water Management District; Michael Link, City of St. Petersburg; Sandra McDonald, Nancy Page, and David Talhouk, Pinellas County; Rick Powers, Bromwell Carrier, Inc.; Jim Cockerill and Art Hill, CDS Technologies, Inc.

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Retrofit Study for the Lower Neshaminy Creek Watershed

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The Lower Neshaminy Creek watershed is a densely developed area in the Bucks County, Pennsylvania, coastal zone. Historically, stormwater best management practices (BMPs) consisted of stormwater management basins designed only to control flooding. Water quality was a secondary concern. Retrofit options to meet the water quality control requirements of section 6217 of the Coastal Zone Act Reauthorization Amendments (CZARA) have been developed. Fifteen basins having different design features and draining different land uses were selected as examples. Retrofit recommendations focus on low-cost design and maintenance options that landowners or local governments can implement. Graphics were developed that demonstrate the problems and illustrate the retrofit plans for each basin. The Bucks County Planning Commission plans to fund the modifications to one basin as a demonstration project.

The hydrologic and water quality benefits associated with a range of additional BMPs that are appropriate to Bucks County were also evaluated. A matrix summarizing the water quality benefits of these BMPs was developed. Implementation costs and maintenance requirements were analyzed. An easily understandable "how-to" document targeted at local government officials and staff, developers, and people who maintain stormwater management facilities was developed to present the information on retrofit options and costs, maintenance requirements, and BMP selection.

Introduction

In July 1995 the Bucks County Planning Commission (BCPC) entered into a grant agreement with the Pennsylvania Department of Environmental Protection (DEP) and the National Oceanic and Atmospheric Administration (NOAA) to complete a study involving modifying options for existing stormwater management basins in the Lower Neshaminy Creek watershed. The study focused on stormwater basins and their effect on stream water quality. By identifying problems and possible solutions for these basins, the BCPC hoped to develop guidance that could be used by local municipalities to repair malfunctioning basins or to learn to design better ones in the future. Guidance was also developed on the selection of appropriate BMPs for different development situations and the cost/benefit of selected BMPs.

Study Area

The predominant land cover in the Lower Neshaminy study area is single-family residential housing (2 to 4 dwelling units per acre). Other significant amounts of land cover in the area include multifamily residential buildings (>4 units per acre) and commercial and manufacturing structures. Minor land cover in the area includes transportation, utilities, community service, military, recreation, agriculture, mining, and vacant land. Projected future land uses include predominantly high-density residential, as well as commercial, industrial, and resource protection.

Retrofit Options for Existing Stormwater Basins

Some residential and commercial developments in the Lower Neshaminy Creek watershed have been built with one or several runoff controls on site. Most of these facilities were designed to control flooding and other hydrologic impacts from development, with little consideration for water quality control. These basins could be redesigned and retrofitted to address both the water quality and water quantity concerns in the watershed.

Stormwater Management Basins Used in the Study

Seventeen stormwater management basins in the Lower Neshaminy Creek watershed were assessed for potential

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retrofitting/redesign options (Figure 1). The survey of basin conditions was conducted during several site visits to the watershed during February through July of 1995. The immediate surrounding areas were also observed for problems that might have been caused by the basins, such as flooding or eroded conditions. General information regarding the physical appearance and condition of the basins, and problems including erosion, improper maintenance, and malfunctioning basin components, were noted.

Basin Summaries

After the site visits and review of the available site plans, summaries were compiled to provide a general description of the conditions at each basin. Table 1, gives examples of basin descriptive information, and the problems and suggestions for remediation. Figure 2 presents the conditions at an example basin and the types of retrofit options developed. Table 2 is a summary of the problems observed in the basins and potential solutions.

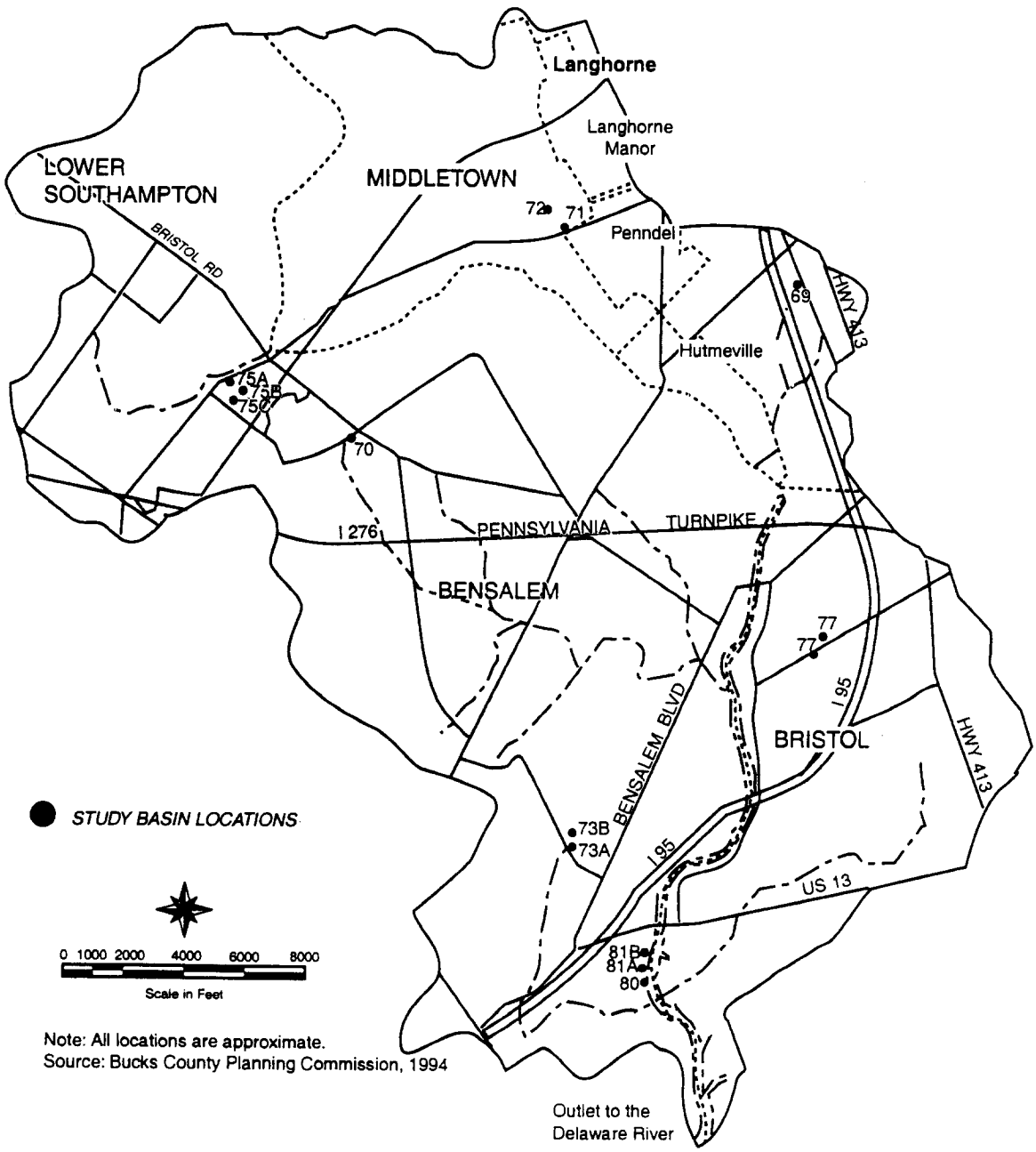

Figure 1. Map of study area, Lower Neshaminy Creek, stormwater management and water quality study.

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Table 1. Example Basin Summaries

Basin	Description	Problems	Suggestions
Basin 70 - Neshaminy Square Shopping Center	• Three-stage outlet structure • Basin appears to pick up runoff from all of parking lot and entrance road as well as rooftops (downspouts discharge to surface)	• Significant erosion at far inlet manhole and catch basin at parking lot • Not well maintained: a lot of trash and discard items • Fairly steep side slopes with dead brush at several locations, very marshy on bottom • No water quality component • Algae present in both the basin and the discharge channel	• Perform maintenance (e.g., fix erosion, clean debris) • Add orifice plate* • Excavate the basin and make it into a pond • Add forebays at each inlet • Add an upstream basin • Regrade side slopes where possible and add vegetation • Install oil and grit separators at existing manholes • Install water quality or sand filter inlets*
Basin 72 - Highland Avenue	• Perforated riser; designed for water quality attenuation • Outlets to street system • Development very new • Concrete overflow spillway	• Poorly maintained basin with poor growth of grass • Needs to be regraded since invert of outlet is higher than portions of basin (causing ponding to occur) • Very steep slopes (slopes should be less for water quality purposes.)	• Regrade and reseed the slope • Plant additional vegetation • Modify slopes; perhaps add terracing
Basin 76 - Old Lincoln Highway	• Cigar-shaped (long and narrow) with steep longitudinal slope • Good stand of grass • Approximately 8-inch diameter outlet pipe • Purpose unknown; majority of parking lot bypasses basin and there is no evidence of roof downspouts	• Erosion around riprap at inlet • No water quality component	• Add orifice plate* • Add forebay at inlet • Intercept parking lot that bypasses the basin • Add vegetation • Add water quality inlets* • Add series of riprap or timber checkdams
Basin 77 - Bensalem Township Industrial Park I	• Large outlet pipe • Outlet to enclosed system; riprap overflow spillway	• Minor amount of marshy bottom • Inlet and outlet adjacent to each other (short-circuit) • No water quality component	• Add orifice plate* • Regrade or add forebays to minimize short-circuiting • Add vegetation • Add water quality inlets*
Basin 78 - Bensalem Industrial Park II	• Steep side slopes • Unconventional shape - long and linear with bulb at top • Two-stage outlet • Parking areas and rooftops drain to the basin	• Partially clogged inlets with some ponding (both typical) • No water quality component • Many tall weeds at bottom, which inhibit drainage • Some erosion on side slopes	• Perform routine maintenance • Add forebays to minimize short circuiting at inlet 2 • Regrade where possible • Add water quality inlets and/or orifice plate*

*Flood protection from large storms may be lessened when orifice plates and/or water quality inlets and outlets are installed.

It is important to note that the observations and the recommended renovations to these facilities or sites were offered for voluntary adoption by municipalities and facility owners. The retrofit procedures described in the report were not intended to be mandatory. The municipal officials and facility owners were strongly encouraged to implement any and all of the measures identified for the upgrades through mutual agreement or cooperative effort. The result of this implementation might be to reduce some types of basin failures (e.g., outlet failure, side slope failure, scouring, standing water in dry facilities), increase overall water quality associated with stormwater runoff in urban areas, and reduce the need for long-term rehabilitation or repair of these facilities.

Costs and Benefits Associated with Urban Runoff Controls

Costs of nonpoint source controls vary from site to site and area to area. Because of the variety of options available for controlling urban runoff, it is difficult to pinpoint exact costs for runoff control. An important factor that needs to be considered in determining such costs is the cost of maintaining facilities. Volume 2 of the *Neshaminy Creek Nonpoint Pollution and Wetlands Study* (Bucks County Planning Commission, 1994) presents information on the relative cost and benefit of various nonpoint source control practices. EPA's *Guidance Specifying Management Measures for Sources of Nonpoint Pollution in Coastal Waters* (1993) also presents detailed information on rela-

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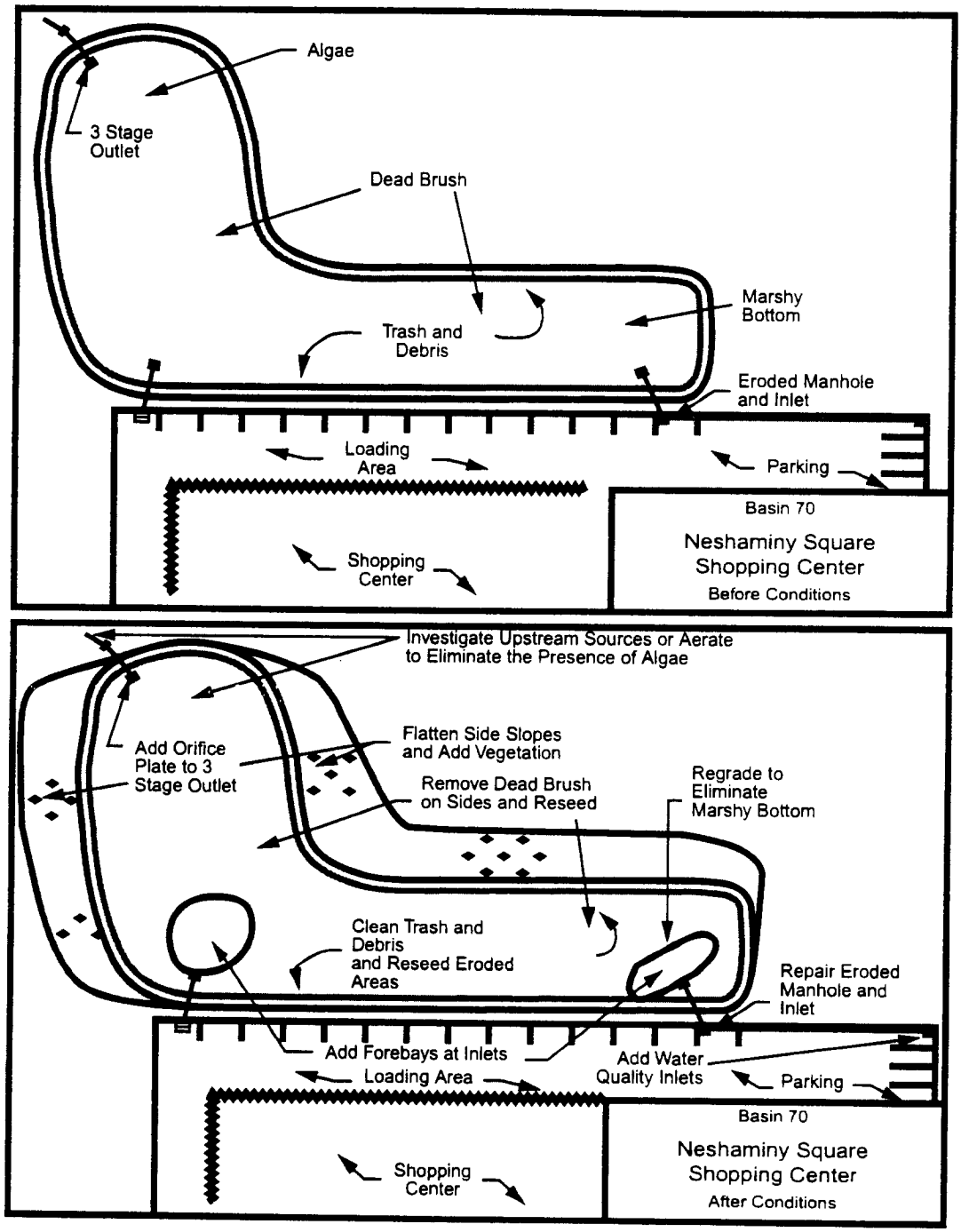

Figure 2. Before and after retrofit conditions at an example basin.

Table 2. Summary of General Problems and Solutions

Problem	Possible Solution(s)
Slide slope erosion	• Regrade slopes • Add vegetation • Install energy dissipators and/or level spreaders
Erosion around outlet structure	• Install energy dissipators/level spreaders
Receiving stream erosion	• Install riprap or similar stabilizing structure
Lack of water quality components*	• Orifice plates • Perforated riser • Sediment forebay • Water quality inlets • Vegetation • Pollution prevention
Trash	• Periodic maintenance • Trash screens
Ponding	• Regrade basins • Reset outlets where appropriate • Plant wetlands species
Sedimentation	• Sediment forebays

*Flood protection from large storms may be lessened when water quality controls are installed.

tive costs for urban runoff controls. Volume II of the *Neshaminy Creek Watershed Stormwater Management Plan* includes information on the maintenance costs associated with various stormwater management practices.

General Costs and Benefits Associated with Urban BMPs

The cost of design and installation is just a portion of the overall cost of implementing structural BMPs. The additional costs that need to be considered include routine maintenance, inspections, modifications if the system is not working properly, and retrofitting, if necessary. It is difficult to anticipate and quantify the exact costs associated with controlling urban runoff.

Cost should not be the only factor in choosing a BMP or series of BMPs. The types of pollutants removed, the feasibility of the BMP in light of site constraints, the amount of maintenance needed, and aesthetics and safety are also considerations. Table 3 provides a general overview of the benefits and disadvantages of typical BMPs, pollutant removal rates, and general construction and annual costs associated with the BMPs. Table 4 summarizes the feasibility and comparative costs for each of the BMPs. These tables are meant as guides only; actual costs may vary.

General Findings

Field assessments were performed on 17 example stormwater management basins in Bucks County, Pennsylvania. Recommendations for modifications or upgrades were applied to the example basins in many forms, including outlet modifications, grading, inlet modificafions, vegetation changes, fencing, and general maintenance procedures. Each basin was documented in "before" and "after" conditions, and changes were discussed in detail. Costs were considered in an attempt to evaluate how "achievable" these modifications could be, assuming that the lower the relative cost, the more likely that the modification could be done. Due to the desire to include a water quality component, the structural modifications were evaluated from the standpoint of how to improve water quality functions. Modifications ranging from costly ($89,700) to reasonable ($10,000) were developed for each individual basin. Information was also presented on the relative costs and feasibility of installing a variety of other BMPs.

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Table 3. BMP Benefits and Costs

BMP	Beneficial for:	Beneficial with some Limitations for:	Advantages	Disadvantages	Pollutants Removed (Average % Efficiency)	General Cost
Extended Detention - Dry Pond	• Flood control • Erosion control	• Water quality	• Provides peak flow control • Provides good particulate removal • Can serve larger developments • Usually does not release warm or anoxic water downstream • Provides excellent protection from downstream erosion • Can create wetland and meadow habitat when landscaped properly	• Removal rates for soluble pollutants are low • Not economical for drainage areas less than 10 acres • If not adequately maintained, can become an eyesore and health hazard • Improper design can lead to significant reduction in efficiency • Extremely large storms tend to "blow through" the system, reducing pollutant removal	• TSS (45%) • Phosphorus (25%) • Nitrogen (30%) • COD (20%) • Lead (50%) • Zinc (20%)	Construction $0.50/ft^3 Annual $0.30/ft^3
Wet Pond	• Flood Control • Erosion control	• Water	• Provides peak flow control • Cost-effective for larger, more intensely developed sites • Enhances aesthetics and can provide recreational benefits • Helps to prevent scour and resuspensions of sediments • Provides good nutrient removal	• May not be economical for drainage areas less than 10 acres • If not properly maintained can become an eyesore and safety and health hazard • Requires considerable space • Not suitable for hydrologic soil groups A and B in the NRCS classification unless the pond is lined, or inappropriate soils are replaced with more appropriate soils • Possibility of release of warm and anoxic water which may impact downstream aquatic life	• TSS (60%) • Phosphorus (45%) • Nitrogen (35%) • COD (40%) • Lead (75%) • Zinc (60%)	Construction (pond < 1 million ft^3) $0.50/ft^3 (pond > 1 million ft^3 $0.25/ft^3 Annual $0.008 - 0.07/ft^3 quality
Vegetated Filter Strip	• Water quality • Erosion control	• Flood	• Low maintenance requirements • Can be used as part of the runoff conveyance system to provide runoff pretreatment • Can reduce particulate pollutant levels in areas where runoff velocity is low to moderate • Provide urban wildlife habitat • Economical	• Can concenterate water, which significantly reduces effectiveness • Ability to remove soluble pollutants is highly variable • Limited feasibility in highly urbanized areas where runoff velocities are high and flow is concentrated • Requires periodic repair, regrading, and sediment removal to prevent channelization	• TSS (65%) • Phosphorus (40%) • Nitrogen (40%) • COD (40%) • Lead (45%) • Zinc (60%)	Construction (*existing vegetation*) $0/acre (*seeding*) $400/acre (*seed and mulch*) $150/acre (*sod*) $11,300/acre Annual - Natural succession (*existing vegetation*) $100/acre (*seed*) $125/acre (*seed and mulch*) $200/acre (*sod*) $700/acre

(continued)

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Table 3. Continued

BMP	Beneficial for:	Beneficial with some Limitations for:	Advantages	Disadvantages	Pollutants Removed (Average % Efficiency)	General Cost
						Annual - No natural succession (*existing vegetation*) $800/acre (*seed*) $825/acre (*seed and mulch*) $900/acre (*sod*) $1400/acre control
Grassed Swale	• Erosion control	• Water quality • Flood control	• Requires minimal land area • Can be used as part of the runoff conveyance system to provide pretreatment • Can provide sufficient runoff control to replace curb and gutter in large-lot single-family residential developments and on highway medians • Economical	• Low pollutant removal rates • Leaching from culverts and fertilized lawns may actually increase the presence of trace metals and nutrients	• TSS (60%) • Phosphorus (20%) • Nitrogen (10%) • COD (25%) • Lead (70%) • Zinc (60%)	Construction (*seed*) $6.50/lin ft (*sod*) $20/lin ft Annual (*seed*) $1/lin ft (*sod*) $2/lin ft
Constructed Wetlands	• Flood control • Erosion control	• Water quality	• Can serve large developments; most cost-effective for larger, more intensely developed sites • Provide peak flow control • Enhance aesthetics and provides recreational benefits • Prevents shoreline erosion • Helps prevent scour and resuspension of solids • High pollutant removal potential	• Not economical for drainage areas less than 10 acres • Potential eyesore and health and safety hazard if not properly maintained • Requires large land area • Possible thermal and anoxic discharge, which could impact downstream aquatic life • May contribute to nutrient loadins during vegetation die-down periods	• TSS (65%) • Phosphorus (25%) • Nitrogen (20%) • COD (50%) • Lead (65%) • Zinc (35%)	Construction $50,000-$100,000/acre (This is based on actual construction costs for a development in northern Delaware)
Sand Filter	• Water quality • Erosion control	• Flood control	• Provides high removal efficiencies of particulates • Requires minimal land area • Provides flexibility to retrofit existing small drainage areas • High removal of nutrients	• Not feasible for drainage areas greater than 5 acres • Feasible only in areas that are stabilized and highly impervious • Not effective as water quality control for intense storms	• TSS (80%) • Phosphorus (60%A) • Nitrogen (35%) • COD (55%) • Lead (80%) • Zinc (65%) • Oil and grease (75%)	Construction $5/ft^3 Annual $0.10 - 0.80/ft^3

Sources include: MWCOG, 1992; Terrene Institute, 1994; USEPA, 1993, 1996.

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Table 4. Relative Costs and Feasibility

BMP	Relative Cost	Feasibility Factors
Extended Detention - Dry Pond	Lowest cost alternative in its size range	• Good if used in conjunction with pretreatment (e.g., sediment forebay, grassed swale) • Requires dedication of land that could otherwise be used for buiding • Viable option if downstream flooding is a concern
Wet Pond	Moderate to high compared to alternatives; however, maintenance requirements tend to be less than with dry ponds	• Provides aesthetic benefits (which could be translated into economic benefits for developers) if creatively designed and properly maintained • Requires dedication of land that could otherwise be used for building • Viable option if downstream flooding is a concern
Vegetated Filter Strip	Low comparative cost	• For use in areas where land can be dedicated for stormwater runoff control • Better for new development than for retrofit in already developed areas • Can be incorporated into the landscape of a development, adding aesthetic value
Grassed Swale	Low compared to curb and gutter	• Requires some maintenance (mowing, cleaning trash) • More aesthetically pleasing than curb and gutter
Constructed Wetlands	Marginally higher than wet ponds	• Provides habitat • Can be used as a selling point for developments • Requires some maintenance (more until wetlands become established) • Should be used in conjunction with other BMPs (e.g., sediment forebay, swales, etc.) to maximize wetland potential
Sand Filters	Comparatively high construction costs; requires regular maintenance	• Disposal of "dirty" sand may be a waste disposal issue in industrial areas because of contents of sand (hydrocarbons, heavy metals, etc.) • Good for use in areas where land is not available for ponds (e.g., retrofit areas) • High TSS removal rate is a definite benefit

Sources include: MWCOG, 1992; Terrene Institute, 1994; US EPA, 1993, 1996.

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The Stormwater Management StormFilter™

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Introduction

Since 1991, the StormFilter™ (formerly the CSF® Stormwater Treatment System) has been treating stormwater runoff from small single retail sites to large urban parking lots, residential streets, urban roadways and freeways. The StormFilter™ is a filtration Best Management Practice (BMP) used for removal of pollutants from stormwater. The flow-through system is housed in concrete vaults utilizing rechargeable filter cartridges which can hold a variety of filter media. An assortment of patented and commercial filter media is available to effectively remove high levels of stormwater pollutants. The appropriate media are selected based on the pollutants expected at the site. The StormFilter™ offers the flexibility of changing to different media if actual pollutant loadings/concentrations differ from expectations.

System configurations include the Precast StormFilter™, the Cast-In-Place StormFilter™ and the Linear Storm Filter™. The precast and linear models utilize pre-engineered precast concrete vaults for ease of design and installation. The precast units can come with traffic-bearing lids for placement in parking lots where they take up virtually no land space. The cast-in-place filters are customized units for larger flows and may be either covered or uncovered underground units.

The StormFilter™ is designed to be especially effective for the treatment of high pollutant concentration flows, and particularly those storms early in the rainy season. In general, the StormFilter™'s efficiency is highest when pollutant concentrations are highest (Lenhart, 1998).

How It Works

The filters work by percolating stormwater through the cartridges containing filter media. The media trap particulates and adsorbs materials such as dissolved metals and hydrocarbons. Surface scum, floating oil and grease are also removed, After passing through the filter media, stormwater flows into a pipe manifold cast into the floor of the vault to an open channel drainage way.

The typical precast StormFilter™ configuration shown in Figure 1 consists of an inlet bay with flow spreader, cartridge bay containing the flow cartridges, an overflow and outlet bay (above outlet).The inlet bay serves as a grit chamber and provides for flow transition into the cartridge bay. The flow spreader provides for the trapping of floatables, oils, and surface scum prior to entry into the cartridge bay. Stormwater enters the cartridge bay through the flow spreader, cascades over an energy dissipater and begins to pond. The StormFilter™ is also designed with an inline overflow which operates when the inflow rate is greater than the flow capacity of the cartridges.

Filter Cartridge Hydraulics

Each cartridge plugged into the underdrain manifold is about 19" tall and 19" in diameter and designed to treat a flow of 15 gpm with 2.3 feet to total system head.

Once stormwater begins to pond in the vault cartridge chamber, it percolates through the media and begins to fill the center drainage tube. As the center drainage tube fills, the air is purged out the one way air relief valve located in the filter top. Once the tube is filled with filtered water, there is enough buoyant force on the float to lift it from its seat. The filtered water is then allowed to flow out of the cartridge into the drainage manifold. As the column of water moves down, the air valve snaps shut and primes a hanging column of water. The hanging column of water then draws the stormwater horizontally through the media and into the inner drainage tube with about 18 inches of suction head. .

The filter cartridge (Figure 2) will continue to draw water throughout the storm duration. At some point during the storm, the outflow rate from the filter will exceed the inflow rate and the water surface elevation in the vault will begin to drop. This will continue until the water surface reaches the lower lip of the hood. At that point the suction head will violently draw air into the hood causing high energy turbulence between the inner surface of the hood and the outer surface of the filter media.

This high energy turbulence scrubs much of the accumulated sediments from the surface of the filter causing 1) a high concentration suspension which then settles, or 2) a direct sloughing of sediment to the vault floor. Sediments

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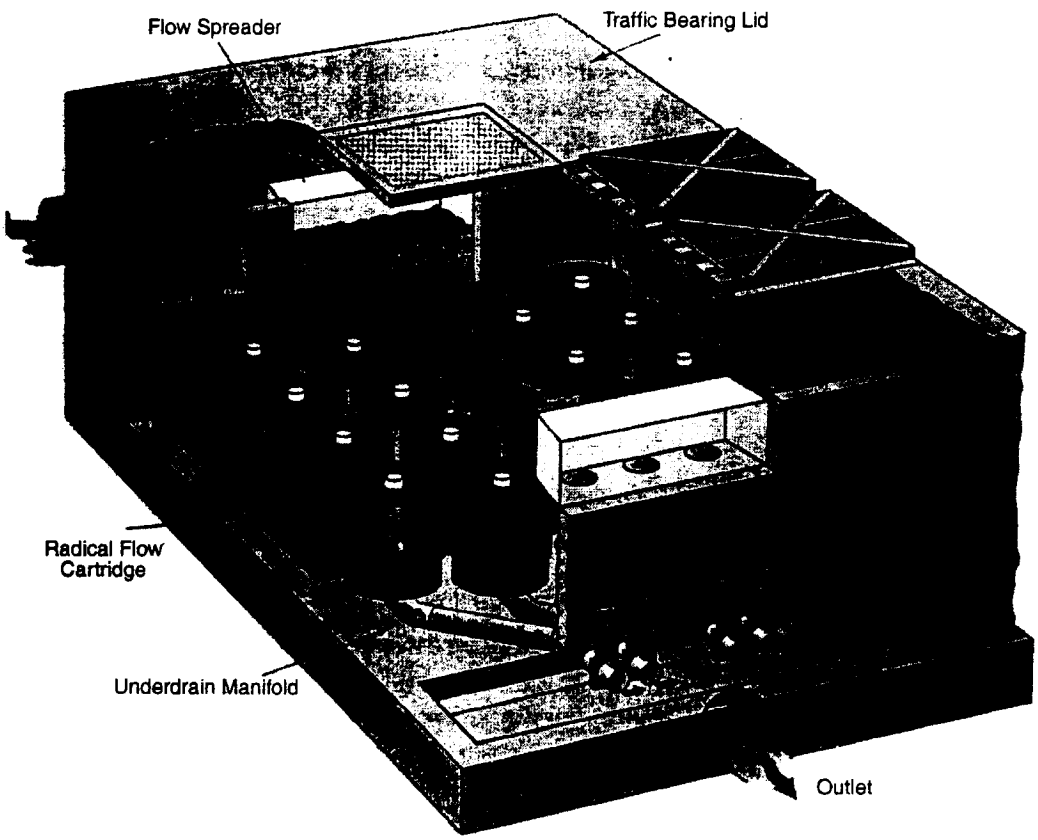

Figure 1. Precast StormFilter™ diagram.

don't get into the underdrain area because all water arriving there has been filtered. This scrubbing action will partially restore the permeability of the filter thus enhancing its performance by increasing the filter life and decreasing filter maintenance frequency and costs.

The Drainage Manifold

The drainage manifold is a pre-manufactured pipe system that is shipped to either a concrete precaster or cast-in-place concrete contractor. The manifold consists of a series of 3"x2" tees on equidistant spacing with control valves at the outlet. The control valves automatically regulate flow to 15 gpm per cartridge. The manifold is secured in place according to plan and cast into the floor of the facility (Figure 3). The removable cartridges are then "plugged" into the manifold tees.

Basic System Design

The StormFilter™ is sized to treat the peak flow of a design storm. Peak flows are typically determined by calculations based on the contributing watershed hydrology and using a design storm magnitude. The design storm is usually based on the requirements set by the local regulatory agency. The size of a StormFilter™ is determined by the number of filter cartridges required to treat the peak stormwater flow.

Each cartridge is designed to treat a peak flow of 15 gpm (or 30 cartridges/cfs). For example: a peak design stormwater flow rate of 150 gpm would require 10 cartridges. The StormFilter™ typically requires 2.3 feet of head differential between the invert of the inlet and the invert of the outlet.

Depending on individual site characteristics, some filters are equipped with high and/or low flow bypasses. High flow bypasses are installed when the calculated peak storm event generates a flow which overcomes the overflow capacity or design capacity of the filter. Base flow bypasses are sometimes installed to prevent continuous inflows caused by ground water seepage.

Available Filter Media

CSF® Leaf Compost Media

In autumn deciduous trees begin to drop their leaves in preparation for the winter. In metropolitan areas the leaves accumulate, clog storm drains, and may cause local flooding. To prevent this, many cities have leaf collection programs. Leaves are either swept up or brought to drop-off points and then transported to landfills, or municipal or commercial composting facilities.

Using a feed stock of pure deciduous leaves (i.e., no mixed yard debris such as prunings and grass), Stormwater

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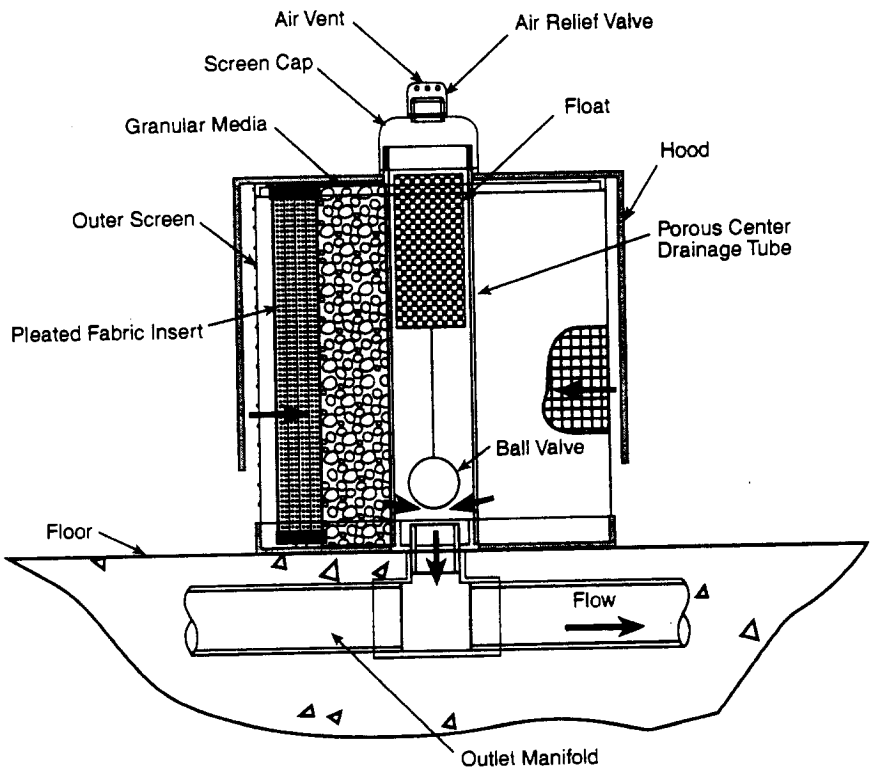

Figure 2. Filter cartridge.

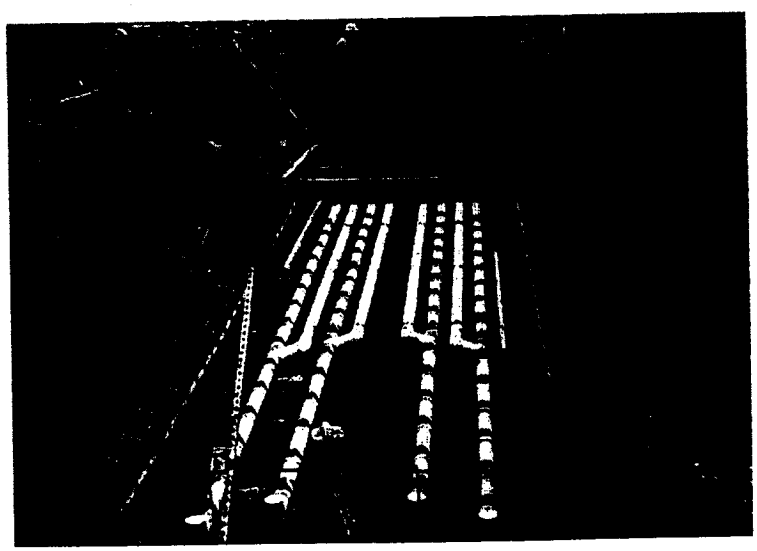

Figure 3. Casting the pipe manifold into the floor of this cast-in-place vault.

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Management composts leaves collected by the City of Portland, OR, over a period of eight months into a mature stable compost. Stormwater Management then processes the finished compost into a granular media which resembles soil and has no odors. Once complete, the medium appears to have the physical and chemical characteristics desirable for the filtration of stormwater.

Pollutant Removal Mechanisms

There are three primary pollutant removal mechanisms performed by the CSF® filter media: mechanical filtration to remove sediments, chemical processes to remove soluble metals, and adsorption properties to remove oils and greases.

Sediments

The media are contained in a series of filter cartridges which have a 7" thick layer of the media through which stormwater passes. Sediments are filtered out on both the surface of the filter and the surfaces of granules throughout the media matrix. As sediments are removed from stormwater runoff and accumulate on the surface of the filter, the permeability will decline thus requiring facility maintenance. Sediment removal will vary with particle size distribution, but removal has been as high as 95%.

Soluble Heavy Metals

The media also acts as a chemical filter to remove dissolved ionic pollutants such as soluble heavy metals, including lead, copper, and zinc. The mechanism of cation exchange is provided by humic acids, which are produced by the aerobic biological activity which occurs during composting. Soluble heavy metal removal rates vary from 65% to 95%.

Oils & Greases

Removal of oils and greases (O&G) and other organic compounds is facilitated by the high organic carbon content of the media. The organic carbon is oleophilic and adsorbs free oil and grease. The system performs best when O&G loadings are less than 20 mg/l. Measured removal rates are as high as 80% (W&H Pacific, 1992).

Pleated Fabric Inserts

The pleated fabric insert is used primarily for sediment control (Figure 4).The insert fits inside the standard cartridge leaving an annular space between the inside of the insert and the drainage tube, which can be used for the addition of granular media to remove selected soluble pollutants.

The reusable insert is constructed of a durable fabric which is easily cleaned with low-pressure water. Each insert has a total of 75 square feet of surface area. Two fabric pore sizes of 70 microns and 36 microns are available. Sediment loading performance data for the two fabrics, using granular media only, are presented in Graph 1.

The CSF® media showed a 50% decrease in flow rate after the accumulation of 12 pounds of dry sediment per cartridge. The 70 μm fabric filter insert prolongs the permeability of the system by three times over media alone and the 36 μm fabric increase the life by a factor of two. As shown by the 50% flow rate decrease line, it would require 23+ pounds of sediment per cartridge to slow the flow rate of the 36 μm fabric to 7.5 gpm and 31+ pounds per cartridge for the 70 μm fabric.

Graph 2 shows the particle size distribution of the sediments used in these tests. TSS removal by the fabric filters is 100% of particles over 36 μm, and some portion of particles under 36 μm (The sediments were taken from existing stormwater quality facilities, dried and screened through a #45 mesh).

Also note on Graph 1, the abrupt jumps in flow rate. These jumps represent the flow recovery generated by the self cleaning mechanism. Once the jump occurred, the sediments that settled to the bottom were intentionally resuspended during testing to ensure that 100% of the sediments used were attached to the filter surface.

Table 1 presents estimated volumes of water treated as well as the approximate dry sediment poundage that a typical 8' x 14' StormFilter™ containing 18 cartridges could treat and capture respectively.

Values in Table 1 assume TSS = 75 mg/l, sediment composed of 50% fine sand and smaller particles, and an 8' x 14' StormFilter™ containing 18 cartridges. The values also assume the removal of 100% of suspended solids with no sediment accumulation at the bottom of the vault.)

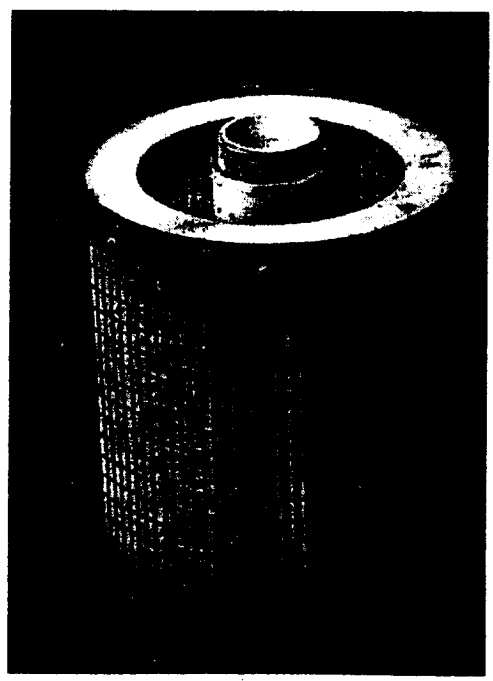

Figure 4. Pleated fabric inserts.

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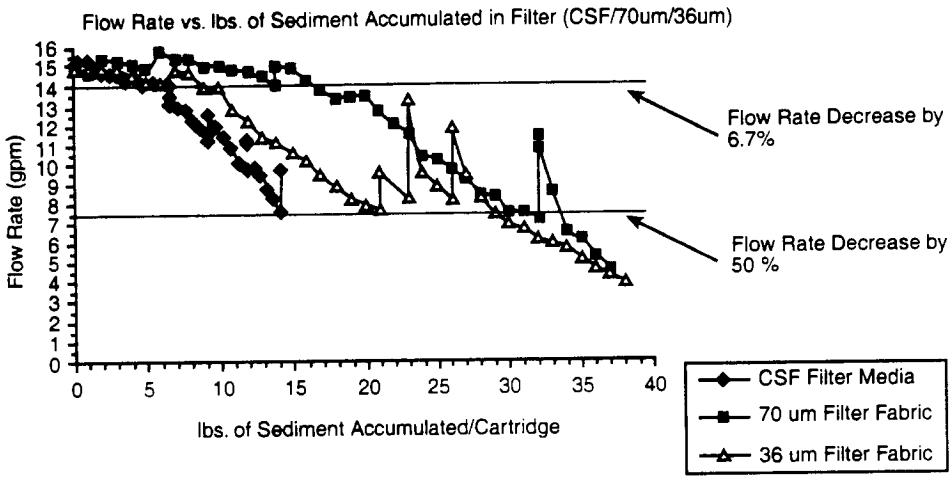
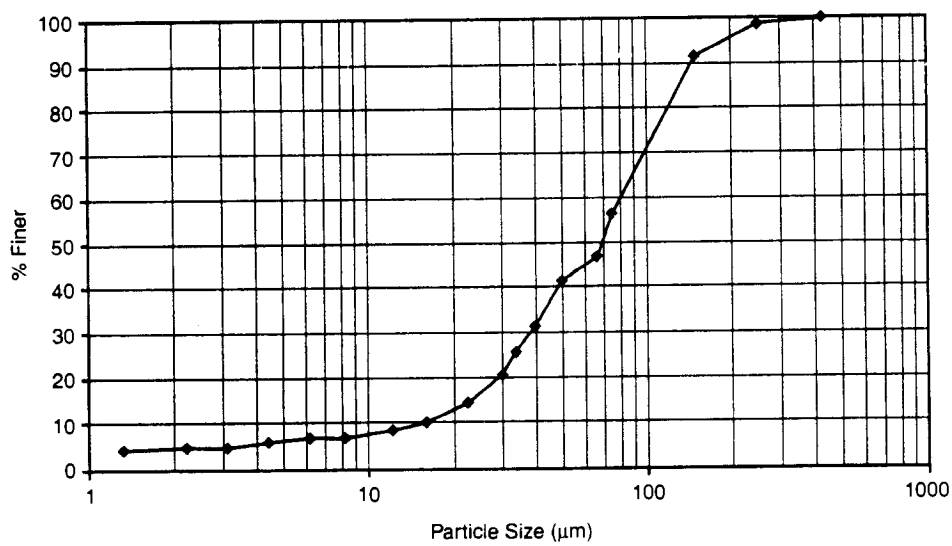


Graph 1. Sediment accumulation reduces flow rates.



Graph 2. 1997 185th particle size distribution.

Table 1. Predicted Sediment Removal Rates

	CSF® Media	36 micron fabric	70 micron fabric
6.7% Decrease (decrease from 15 to 14 gpm)	115 pounds sediment	200 pounds sediment	310 pounds sediment
	180,000 gal. water	320,000 gal. water	500,000 gal. water
50% Decrease (decrease from 15 to 7.5 gpm)	260 pounds sediment	425 pounds sediment	560 pounds sediment
	409,000 gal. water	680,000 gal. water	892,000 gal. water

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If the designer knows the maximum allowable decline in system efficiency and the system is designed for the 15 gpm/cartridge, the graph and table can be used to establish the total sediment loading that occurs between maintenance cycles. (More water has passed through filters when their permeability has decayed to 50% than when it has decreased by only 6.7%.)

Perlite and Zeolite Media

Perlite is a "puffed" volcanic ash. This lightweight material is commonly used as a filter for water filtration. Though not chemically active, Perlite is very effective for removal of fine particles due to its micro pores and blocky structure. Perlite can be used as a stand alone medium or in conjunction with the pleated fabric insert. Perlite can also be mixed with other media such as Granulated Activated Charcoal (GAC), commercial cation exchange media, etc. Perlite would inexpensively act on TSS while GAC, etc. would act on organics.

Zeolites are naturally occurring minerals that exhibit cationic exchange properties. Some zeolites have the capacity for anion adsorption. Blended perlite/zeolite media produced by Stormwater Management is suggested for watersheds where soluble phosphorus is of concern.

To demonstrate the removal efficiency of phosphate by the zeolite, 2-inch column studies have been performed with a varying matrix of zeolite mixed with a highly permeable perlite. The tables below present the data obtained from several column studies and show the percent removal of ortho-phosphate promoted by the zeolite. (Each table is followed by a brief description of the sample number and sample time.) The influent source was irrigation return water from a commercial container nursery.

The samples labeled 10G-1 and 30G-1 represent the first effluent samples taken from a 10-gram and 30-gram column test respectively. These samples were taken after approximately 100 ml of stormwater had passed through each 2-ing diameter column. The samples labeled 10G-2 and 30G-2 represent samples taken after approximately 1000 ml of stormwater had been passed.

Table 2. Ortho-Phosphate Removal

Sample	Influent	Sample 10G-1	Sample 10G-2	Sample 30G-1	Sample 30G-2
Ortho-P (mg/l)	0.33	0.18	0.25	0.15	0.18
% removal		45.4%	24.2%	54.5%	45.4%

Tables 3, 4, and 5 represent preliminary testing of phosphate (at three concentrations) removal.

Table 3. Ortho-Phosphate Removal

Sample	Influent	Sample 40.3	Sample 40.5
Ortho-P (mg/l)	0.37	0.20	0.32
% removal		49.5%	13.5%

The sample labeled 40.3 represents a composite sample of the first 4 gallons of effluent passed through 40 grams of zeolite with the 40.5 sample being the 5th gallon passed.

Table 4. Ortho-Pohsphate Removal

Sample	Influent	Sample 50.1	Sample 50.3	Sample 50.5
Ortho-P (mg/l)	0.38	0.15	0.34	0.36
% removal		60.5%	10.5%	5.3%

The samples labeled 50.1, 50.3, and 50.5 represent samples taken of the 1st gallon, the 3rd gallon, and 5th gallon of effluent respectively.

Table 5. Ortho-Phosphate Removal

Sample	Influent	Sample 60.5
Ortho-P (mg/l)	0.37	0.19
% removal		48.6%

The effluent sample labeled 60.5 represents a combination of the entire 5 gallons of stormWater passed through the column.

Though further studies are underway, these data indicate a sorbtive capacity of 60 mg ortho-phosphate/kg of zeolite media. We expect to achieve double that removal rate soon. Depending on the mass concentration of zeolite in the media, one is able to estimate the mass removal of ortho phosphate between filter maintenance cycles.

Facility Maintenance

The primary purpose of the StormFilter™ is to filter out and prevent pollutants from entering down gradient water bodies. Like any effective filtration system, these pollutants must be periodically removed to restore the StormFilter™ to its full efficiency and effectiveness. Maintenance requirements and frequency are dependent on the pollutant loading characteristics of each site. To assist with maintenance issues, detailed Operation & Maintenance Guidelines are available.

Maintenance services can be purchased completely, or in part. Stormwater Management also provides tracking of all installed systems, notifies system's owner of maintenance needs and notifies the regulatory agency that the system has been maintained.

Maintenance is usually performed late in the dry season to rejuvenate the filter media and prepare the system for the next rainy season. Maintenance activities can also be performed mid-season in the event of a spill or excessive sediment loading due to site erosion.The most common maintenance schedule is once a year, but sites can and do vary.

Maintenance involves replacing the cartridges with a series of newly recharged cartridges. The old cartridges are then cleaned and recharged with new media for reuse. Systems with excess sediment accumulation on the vault floor can be vactored out.

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Media residuals can be re-composted to reduce accumulated hydrocarbons and then used in landscaping, erosion control applications or daily cover for landfills. This sustainable process helps minimize total maintenance costs.

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Bioretention: An Efficient, Cost Effective Stormwater Management Practice

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Abstract

In 1993 the Department of the Environmental Resources, Prince Georges County, Maryland, introduced the "Design Manual for Use of Bioretention in Stormwater Management". Bioretention facilities have become commonly referred to as rain gardens. They were designed as an alternative, cost effective stormwater best management practice (BMP) which allows multifunctional use of green space and landscaped areas for storage and treatment of runoff. Bioretention BMPs are simply very shallow landscaped depressions where runoff is infiltrated or filtered through a soil/plant complex for treatment. Bioretention has been successfully used as an alternative, cost-effective BMP for commercial, industrial and residential applications addressing both landscape and stormwater management objectives. More recently, as a retrofit practice, the BMP has been incorporated into green space, streetscapes, median strips and parking lot islands. This paper summarizes recently completed monitoring findings and the lessons learned over the last five years with the use of bioretention for urban stormwater management.

General Bioretention Design Features

Bioretention was originally modeled after the hydrologic/ physical characteristics of an upland terrestrial forest or meadow community (as opposed to a wetland) dominated by facultative trees, with understory shrubs and herbaceous upland plant materials. The BMP is strategically placed to intercept drainage and is therefore, subject to repeated hydraulic loading. Because of this, the designer must be sure that the BMP will be well drained to maintain aerobic conditions. Proper drainage can be achieved by infiltration (where soils allow), underdrains, or both. Key factors in the design and construction of bioretention facilities are careful selection of plant material that can tolerate extreme hydrologic regimes; good drainage to prevent anaerobic conditions; safe conveyance of overflows; careful selection and control of backfill soils; and, careful inlet/ outlet controls to prevent erosion.

Bioretention BMPs should be designed as "off-line" control devices where excess runoff and high-velocity flows bypass the BMP minimizing erosion and flushing of landscape materials and debris. Stored runoff (ponded water over the bioretention area) should exfiltrate over a period of less than a day into the underlying soils and in some cases, into an underdrain that discharges to an appropriate conveyance system. Where soil infiltration rates are lower than 1 inch/hour, or in order to extend the life of the facility, underdrains should be used.

Bioretention areas typically consist of the following components: shallow ponding area (6" or less), mulch layer (2-3"), sandy planting soil (2-3'), a variety of plant materials and where appropriate underdrains. The design of the BMP can vary greatly to accommodate site constraints, ground water recharge, soils, habitat/ecological objectives, watershed hydrology and aesthetics. The freedom in design variation creates opportunities for site integration of natural features with man-made infrastructure. Specific configuration and location is determined after site constraints such as location of utilities, groundwater level, steep slopes, underlying soils, existing vegetation and drainage are considered.

The drainage area for one facility should generally be between 0.25 and 1 acre. Multiple bioretention facilities are needed for larger drainage areas. The storage volume of the facility will be determined by the desired level of control (e.g. first half-inch of runoff) and dewatering capabilities of the design. The BMP works best when there are many facilities with small drainage areas. Large facilities with large drainage areas tend to allow soils to remain saturated creating anaerobic conditions, stressing the plants and potentially reducing the pollutant removal effectiveness for many pollutants.

Planting soil should be sandy loam, loamy sand or loam texture and have clay content of 10% to 15% or less. The pH of the soil should be between 5.5 and 6.5 to maximize

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plant growth and microbial activity for the up-take and transformation of pollutants. The planting soil should contain 3 to 5% organic content and other soil augments typically needed to support the types of plants used.

Native plant species are recommended as they are generally more suited to regional climate, soils, hydrology, and disease. The designer should assess aesthetics, site layout, habitat objectives and maintenance requirements when selecting plant species. After placing the trees and shrubs, the ground cover and/or mulch should be established. Ground cover such as grasses, legumes or flowers can be used. Two or three inches of commercially available fine shredded hardwood mulch or shredded hardwood chips should be applied to provide protection from erosion, as well as to enhance evapotransporation in the facility.

Monitoring

In an effort to refine bioretention design and determine the pollutant removal efficiency of the BMP, a two-year study was conducted by the University of Maryland, in conjunction with Prince George's County, Department of Environmental Resources and the National Science Foundation. The study, known as the "Optimization of Bioretention" and included both laboratory and field testing.

The laboratory experiments were conducted in two small (3 ft x 2.5 ft x 2.5 ft deep) and one large (10 ft x 5 ft x 3.5 ft deep) bioretention "boxes". These boxes were filled with a sandy loam soil topped with a 1-inch layer of commercially available shredded bark mulch and planted with creeping junipers. Perforated pipe sampling ports were placed at three depths (upper, middle and lower) within the larger box to examine pollutant removal as a function of depth and time of exposure. Two sampling depths were used for the smaller box. Effluent samples were analyzed in the University of Maryland's Environmental Engineering Laboratory.

A synthetic runoff recipe was used for all testing to allow for greater consistency and correlation of field and laboratory results. The runoff recipe was based on the average pollutant loading taken from the county's 3-year wet weather data collected as part of the NPDES stormwater monitoring program. The hydraulic loading or rate of application to the bioretention facilities was based on a typical rainfall event in Prince Georges County (0.1 in/hr with a 6-hr duration).

Average removals in the laboratory bioretention systems are shown in Table 1. Sample contamination resulted in no nitrate data for the small boxes. In both cases, removal of the heavy metals (copper, lead, and zinc) was excellent, ranging from 93-99%. Effluent levels of phosphorus showed 70-80% removal. Average TKN and ammonia removals were 60-80% in the lower effluent ports, but significant ranges were noted for each. The system removed only a small amount of the nitrate (23%) and nitrate concentrations above the influent levels were noted from the upper ports.

For the small bioretention boxes, flow from the bottom began approximately 45 minutes after the runoff application. It took about 2 hours for flow to occur from the bottom ports in the large box. In both boxes, the head built up to 3-5.5 inches.

Field Experiment

Field monitoring was performed on an existing bioretention facility. This facility was constructed with an underdrain at a depth of about 2.5 ft., which was used for the sampling port. The same runoff pollutant "recipe" was applied in the field. The runoff application continued for nearly six hours with samples taken every 25-30 minutes.

Results from the field experiment mirrored the laboratory results. For copper, lead, and zinc, nearly total removal was achieved by the bioretention facility with removals exceeding 95%. All metal concentrations in the effluent were less than or near instrument detection limits (2 mg/L for copper and lead, 25 mg/L for zinc). The total phosphorus removal was about 60%. TKN removal was about 50%. The ammonia removal was somewhat variable but the majority of the samples monitored indicate about 70% removal. The removal for nitrate was only about 15% with wide variation.

Summary of Monitoring Observations and Findings

Heavy metal removal showed excellent agreement between the field and laboratory experiments. Removals were

Table 1. Summary of Percent Removal Rates for Laboratory Bioretention Systems

	Cu (µg/L)	Pb (µg/L)	Zn (µg/L)	P (mg/L)	TKN (mg/L)	NH_4^+ (mg/L)	NO_3^- (mg/L)	TN (mg/L)
Large Box								
U	90	93	87	0	37	54	(-97)	(-29)
M	93	99	98	73	60	86	(-194)	0
L	93	99	99	81	68	79	23	43
Small Box								
U	91	95	93	16	55	(-7)	-	-
L	97	98	97	70	76	60	-	-

Testing Conducted by the University of Maryland, Department of Engineering

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greater than 90% and variations were small. Nearly the entire metal uptake occurred within the top few inches of the bioretention system. Separate small column laboratory experiments with the mulch showed that it has a high capacity for metals uptake. It is likely that within bioretention facilities, significant metal uptake occurs within the surface mulch layer. Additionally, examination of the laboratory bioretention mulch after several applications of the synthetic runoff showed elevated metal levels. All results point to the importance of the mulch layer for metals uptake in bioretention.

Phosphorus removal appears to have linear correlation to depth, with better removal resulting as runoff migrates to deeper levels. Limited additional removal was observed beyond depths of about 2 to 3 feet.

TKN removal averaged about 60% with significant variability. The depth of the facility does not appear to significantly affect TKN removal rates.

Ammonia removal appears to increase to over 70% at greater depths. However, due to significant variation in results, depth correlation is difficult to summarize.

Nitrate removal results were erratic and significant variability was found. Shallow sampling ports showed nitrate levels higher than the input, indicating conversion of organic nitrogen, or ammonia to nitrate during the runoff event, or washout of nitrate from previously captured nitrogen. At depths greater than 30 inches, limited nitrate removal occurred (15-20%).

Additional research is needed to optimize the pollutant removal/transformation capabilities of the plant soil complex. It appears possible that the chemical make-up of the soil complex and the types of plants used for bioretention can be customized to address unique pollutant runoff problems and recharge needs for a given land use and watershed.

Bioretention Design & Construction Considerations Based on Monitoring Results

Laboratory and field studies, along with field observations over the last five years have provided data necessary to refine bioretention design and construction criteria. Modifications to the Prince Georges County bioretention design manual are currently underway based on these findings. Some of the major modifications are listed and briefly described below:

Optimizing Facility Depth - The original design depth was set at 4 feet. This was to ensure adequate depth for plant growth and to maximize pollutant removal. The experiments performed to date suggest that sufficient removal is achieved at a 2 -2.5 foot depth. Therefore, design depths can be reduced without significantly compromising pollutant removal efficiency. This will result in cost savings by reducing excavation and back-fill material costs. Underdrains are recommended for all facilities unless geotechnical reports indicate otherwise.

Minimizing Above Ground Storage Depth - Above ground storage of runoff is necessary to achieve retention storage for treatment and management of stormwater runoff. However, the depth of the runoff should be minimized to allow quicker infiltration and evapotranspiration. This will ensure adequate aeration of soils to maintain aerobic conditions necessary for plant growth and various pollutant transformation processes. The recommended maximum ponding depth is 3-6 inches per facility.

Minimizing Hydraulic Loading - The drainage area for bioretention BMPs should be minimized to reduce hydraulic loading of the facility and avoid excessive saturation of the soils which could lead to anaerobic conditions. The maximum recommended drainage area is 1 acre. Ideally, facilities should be designed to control 0.25 acres or less if site/soil conditions and land use allow.

Planting Soil and Soil Amendments - Soil amendments are only necessary for plant viability. The planting soils should have no greater than 10-15% clay particles. Soil infiltration rates of greater than 1 inch an hour are preferable if no underdrain system is used. Native plants require less maintenance.

Facility Sizing and Configuration - Sizing of the facility is dependent upon the drainage area, land use and site constraints. On-lot facilities are limited to space available and grading. As previously indicated, facilities may have soil depths of 2 feet and still achieve significant pollutant removal rates. In addition, the side walls do not need to be straight up and down, but can be shaped in a "dish arrangement" beginning at the ground level around the perimeter and extending to a the design depth in the center. All facilities should be "off-line" to avoid pass-through waters that could cause erosion and flushing of debris from the facility unless carefully designed to accomodate these effects.

Settlement and Compaction- During construction, due to the nature of the planting material and method of installation, settlement will occur. In the University Maryland experiment, 15-20% settlement of the soil was observed after the first influent was applied. Interestingly, after the initial settlement, only minor settlement was reported. In construction, settlement is a concern particularly for the accuracy of final grading and volume storage. Settlement has implications related not only to construction method, but cost as well. To help control problems created by differential or unexpected settlement, wetting of the planting soil prior to placement of mulch and plant materials is recommended. This can be accomplished by hosing down the area (or if time permits, waiting for a rain event). Compaction of the soils must be minimized during construction activities. Bioretention facilities that are shallow (2-3') can be constructed and dressed using manual labor and small machinery.

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Planting Arrangements- Planting arrangements are important for aesthetics, soil stability, habitat, pollutant uptake and the viability of the bioretention facility. Initial planting arrangements should be designed to provide an immediate dense and perpetual ground cover. For example, use of ornamental grasses provides thick, quick growing cover and uptake of pollutants. Facilities located in areas of high visibility should include plant material that is aesthetically pleasing with year-round interest. The use of native planting arrangements is encouraged. Designers should work closely with landscape architects and nurserymen to plan an aesthetically pleasing, yet cost conscious product. Plant materials and landscaping can affect the cost of the facility dramatically accounting for as much as 60% of the total facility cost.

Designing for Thermal Attenuation - Thermal pollution in runoff to receiving waters has been correlated to increases in urban development. Heated runoff temperatures from impervious surfaces is one of the main causes of elevated stream temperatures. By directing and capturing heated runoff into a bioretention facility to allow evaporation, infiltration and filtration into the soil, the facility can act as a heat sink and dissapation. Where protection of cold water fisheries is an important consideration, bioretention can be used to reduce thermal impacts of urban runoff.

Designing for Hydraulic Conductivity - When designing and locating bioretention facilities without underdrains, special attention must be given to the hydraulic conductivity of the surrounding in situ soils. They should have a very high percentage of sand particles (2mm in size or greater) to ensure adequate infiltration. Soils must be USDA classification sandy loam or better in the textural triangle. When performing feasibility analysis for siting bioretention facilities, the hydraulic conductivity of the surrounding soils must be analyzed sufficiently by geotechnical means such as soil borings. Soils investigations typical to the requirements for standard infiltration trenches can be performed to determine the suitability of the in situ soils. Soils having a significant percentage of small soil particles can lead to failures. In addition, even where boring samples indicate small percentages of clay, the soils analysis should determine the makeup of the soil strata and horizons to ensure that clay lenses are not present.

Bioretention Applications

Bioretention systems maximize the use of natural physical, chemical and biological pollutant removal and transformation processes to treat runoff. They are dynamic living micro-ecological systems that demonstrate how the landscape functions to protect the integrity of a watershed's aquatic and riparian ecosystems. Their designs also demonstrate the interconnections of a wide array of environmental and engineering principles and disciplines including: the hydrologic cycle, nonpoint pollutant treatment, nutrient cycles, resource conservation, habitat creation, soil chemistry, ecology, horticulture and landscape architecture.

Bioretention systems use upland facultative plants that are not dependent on a constant source of water such as conventional stormwater /wetland ponds. Therefore, bioretention systems can be constructed in the upland areas avoiding destruction of riparian buffers and streams. Typically, upland soils are more conducive to the natural properties that bioretention facilities are attempting to mimic. Those same soils often are associated with hydrologic soil groups "A" and "B", which exemplify well drained soils.

Widespread use and uniform distribution of bioretention storage throughout a development can also help to replicate predevelopment watershed hydrologic functions. This BMP can be used to reproduce similar rainfall storage capabilities in the developed site that existing prior to development. Retention and groundwater recharge functions are designed to mimic predevelopment runoff characteristics. Reproducing predevelopment hydrologic functions can be the most important factor in maintaining the ecological integrity of receiving waters, small streams and wetland systems.

To achieve uniform distribution of bioretention practices for the greatest hydrological benefits and to maximize pollutant removal, it is important to understand how to integrate the practice into the developed landscape. Below is a listing and brief discussion of the possible ways to apply bioretention throughout a site.

Incorporated into Parking Lots, Medians and Landscape Islands - The design must ensure that infiltration and ground water seepage will not adversely affect the structural integrity of roadways or buildings. Careful grading, location and use of underdrains can minimize problems. It is important to divert overflows to inlets or grass areas in order to prevent deposits of sediment and debris onto parking surfaces.

Designed into Existing Meadow or Forested Areas - These areas can be converted to rain gardens by constructing small earthen or stone berms (4 inches or less) to allow shallow ponding. Care must be taken not to pond too much for too long, as existing mature vegetation may be less tolerant to drastic changes in hydrology. Also, adequate measures must be taken to reduce erosion potential when directing increased volumes and concentrated flows into existing vegetated areas.

Fringe Forest and Transition Areas - Bioretention can be used for re-vegetation of forest fringe areas to recreate a terrestrial forest community and transitional habitat ecosystem. These areas would consist of trees, sub-canopy under story trees, a shrub layer and herbaceous ground covers. Plants can be selected for their habitat value (food, shelter and nesting materials). Deep rooted vegetation would help to promote increased infiltration.

Open Space Meadows - Open space areas not used for recreation or other purposes can be designed as rain gardens. Where soils and topography allow, wild flower meadow basins can be constructed. Care must be taken to prevent erosion and to disperse flows throughout the bottom of the rain garden basin.

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Open Swales - Rain gardens must be carefully designed to be used on-line in an open swale. They may be used as an appurtenance to a swale in an off-line configuration. On-line systems are subject to erosion due to high velocities and concentrated flows without the careful design.

Landscape Trees and Shrubs - A simple application of a rain garden is to create shallow depression storage areas around each individual landscape plant. Careful selection of water tolerant species could allow ponding depths of 2 to 3 inches around each plant where soils allow.

Retrofit Existing Development - Many green spaces and landscaped areas can be converted to rain gardens. The applicability of retrofit options will be dependent on a detailed site evaluation.

Sediment Control - Like stormwater management ponds, bioretention pits may be utilized for sediment control devices for stabilization during construction activities. By using on-lot bioretention for sediment control compliance, the need to drain the site runoff to one large holding pond is eliminated or diminished.

Limitations

Bioretention that relies on infiltration alone for dewatering should not be considered where the water table is within 6 feet of the ground surface and when the surrounding soil is unsuitable for infiltration (less than one inch/hour). While the bioretention concept relies on the natural and physical properties of infiltration, absorption and evapotransporation, these processes can have limited capacity under various conditions such as saturated soil, frozen ground, or high humidity. The practice is also not recommended for areas with steep slopes greater than 25% or in areas where extensive tree removal would be required.

When used on residential lots to fulfill stormwater requirements, property owners must be educated on the need and routine care of rain garden areas. Maintenance agreements, educational materials and easements are possible ways to ensure long term use and operation by the property owner.

General Cost Comparisons

Bioretention costs are most attractive when compared to the use of other structural BMPs such as ponds. Cost savings over conventional BMPs can vary widely depending on unique site conditions. More efficient use of land can be achieved when integrated into the typical landscape features. Savings of 10 to 25% compared to conventional approaches have been achieved using rain gardens in residential and commercial sites. Generally, residential rain gardens will average about $3 to $4 a square foot depending on soil conditions and density and types of plants. Commercial/industrial sites costs can range between $10 to $40 a square foot depending on the density / types or plants and the need for control structures, curbing, storm drains and underdrains. Planting costs can vary substantially and can account for a significant portion of the facility cost. In many cases, bioretention can be an extremely cost-effective practice for controlling stormwater. Replacing traditional piping with gardens to convey flow can lead to substantial savings.

There are additional costs when compared to typical landscaping treatment due to the increased number of plantings, additional soil excavation, backfill material and use of underdrains. The use of a wildflower meadow rain garden to replace open space turf will have higher site preparation and planting costs, but long-term maintenance (mowing) can be reduced to once a year.

Long Term Maintenance Considerations

Rain gardens require routine periodic maintenance (e.g. mulching, plant replacement, pruning and weeding) typical of any landscaped area. No special maintenance equipment is needed. Routine maintenance costs will increase proportionately to the increased number of plants used and the area planted. To date, there is no solid data on the longevity of bioretention systems. The use of underdrains will help to ensure the BMP remains well drained for the long term health of the plants. Underdrains placed at shallow depths (2 to 3 feet) can be easily maintained if clogged by sediment or roots.

References

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Low Impact Development Design Manual (PGDER, 1997).

Native Trees, Shrubs, and Vines for Urban and Rural America (Hightshoe, 1988).

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Innovative Stormwater Treatment in Washington State

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Introduction

The Washington State Department of Transportation (WSDOT) has a long history of engineering innovation. WSDOT built the first floating concrete pontoon bridge in the world. Other firsts have been achieved in avalanche control and de-icing treatment.

Not every new technology works as theorized. New systems often need to be fine tuned before they can become a standard practice. Innovative Best Management Practices (BMPs), a continuation of WSDOT's engineering innovation tradition, provide design engineers with more appropriate and effective stormwater treatment options. An innovative BMP is one that has not been approved by the Washington Department of Ecology, our state environmental regulatory agency.

WSDOT maintains a Stormwater Management Program (SWMP) to protect water quality through the National Pollutant Discharge Elimination System (NPDES) municipal permit requirements. The permit requires WSDOT to:

- Reduce and control discharge of pollutants to the maximum extent practicable, as required by federal regulations;
- Use all known, available, and reasonable methods of prevention, control, and treatment.

Innovative BMP research is a critical element of our Stormwater Management Program. State and local governments rely on WSDOT to protect and maintain existing water quality. WSDOT does this by using BMPs.

Standard BMPs

WSDOT's Highway Runoff Manual (HRM) provides uniform technical stormwater management guidelines for our highway designers and other stormwater professionals. Stormwater treatment is required when 5,000 square feet of new impervious surface is added to the highway foot print. Design guidelines for standard BMPs are in the HRM and are approved by the Department of Ecology. They include: biofiltration swale, wet pond, infiltration pond, wet vaults, and nutrient control wet ponds.

Innovative BMPs

Oftentimes design and hydraulic engineers cannot select standard BMPs because:

- Technology is constantly changing and the science of stormwater treatment is advancing.
- There is a need to comply with more stringent local requirements.
- Space is limited and expensive.
- Land is not available to install a conventional BMP.
- Land has slope, soil, or light problems.
- A specific water quality need is required which a standard BMP can not satisfy.

These limitations have created a need for more adaptive and effective BMPs. WSDOT has responded by developing an Innovative BMP Development and Research Program. After an innovative BMP performance has been verified and accepted by the Department of Ecology, the BMP will be added to the HRM — if it performed successfully.

Documenting Innovative BMP Performance

Monitoring innovative BMP sites provides valuable performance data. WSDOT's Stormwater Management Program defines our stormwater research protocols and procedures. The SWMP can be downloaded through the

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Internet from WSDOT's WEB page. WSDOT has determined that the primary pollutants of concern that are representative of highway runoff are:

Solids:	Total Suspended Solids	Total Dissolved Solids
Metals:	Cadmium (total) Copper (total) Lead (total) Zinc (total)	Cadmium (dissolved) Copper (dissolved) Lead (dissolved) Zinc (dissolved)
Oxygen Demand:	Biochemical Oxygen Demand (5 day)	Chemical Oxygen Demand
Nutrients:	Phosphorous (total) Orthophosphates	Nitrate-Nitrogen

These stormwater constituents will be analyzed routinely at our research sites. On a semi-annual basis, WSDOT performs priority pollutant scans which include polynuclear aromatic hydrocarbons, ultimate (20-day) biochemical oxygen demand, and effluent toxicity using the Microtox™ technique.

Vegetated Filter Strip

Research Objective

Vegetative filter strips currently exist along many of our rural highways; however, they are not considered a standard BMP by the Department of Ecology. The research objective is to gather the needed data to demonstrate that a vegetative filter strip should be accepted as a standard BMP in our HRM and under what conditions. Three test filter strips are being evaluated at our SR 8 Black Hills research site in Thurston County.

Design

Vegetative filter strips are planned for use in rural areas on state highways. The average daily traffic count at these sites does not exceed 30,000 vehicles per day. Vegetative filter strips run parallel to the roadway, and runoff from the roadway flows off of the roadway, across the shoulder and then across and into the vegetative filter strip. Typically filter strips are 15 feet wide and 2 to 4 feet from the edge of the pavement.

Figure 1 shows a filter strip BMP plan view site layout of a facility which was built in January 1996. The slot drain serves as the control for runoff volume and pollutant constituents. Three different soil matrixes are evaluated at the research facility, including commercially available compost, organic-rich soil from a local river bottom, and the existing rock and soil that was excavated (or road-Ex) at the research site. Each filter strip was hand seeded with Mechlenberg Fescue.

The facility design allows the amount of runoff moving across and into each filter strip to be monitored, as well as to determine the pollutant removal performance. Because each filter strip was lined with a clay liner, a water balance can be performed. To differentiate between surface overflow and flow through the filter strip, perforated drain pipes were installed at the top and at bottom of each filter strip. The drain pipes are separated by an impervious layer to prevent cross-flow.

Preliminary Results

Data have been collected for twelve storm events over two years. Data will continue to be collected for an additional year. Washington State University evaluates the data and prepares a report.

Preliminary results demonstrate that compost fill may provide stormwater detention as well as treatment. The compost soil matrix captured and held the stormwater; only a small proportion of the total storm event flowed through the compost media. No surface overflow was recorded in any storm event.. The data also show that composted material initially releases nitrogen and phosphorus into the discharge water. This may be a concern in watersheds with sensitive lakes.

The organic soil matrix allowed about half of the runoff to sheet flow over the filter strip and the other half to infiltrate through the soil matrix. The road-Ex soil allowed more than 80% of the runoff to sheet flow over the filter strip and less than 20% of the runoff to infiltrate through the filter strip.

The grass grew best on the compost soil, followed by the organic-rich top soil. Grass grew the poorest on the road-Ex. Because of poor seed germination in the organic-rich soil and road-Ex, grass seed and water was reapplied on three separate occasions. Runoff and infiltration rates were directly related to grass growth. Increased infiltration was observed as grass growth improved.

Difficulties

Accurately recording the discharges through the system was very difficult. Runoff volumes were too small to get a measurable head in the smallest (0.4 HS) commercially available flume. On the other hand, runoff was too large to contain in a single container. In addition, runoff and infiltration ratios varied widely from each filter strip. It was also very difficult to use commercially available flow-proportionate samplers to collect samples for water quality analysis. Through many trials and disappointments, we finally were successful with:

source	quantity measurement	quality measurement
slot drains (control sample)	0.4 HS flume in conjunction with ISCO 3700 samplers	flow proportioned samples using ISCO 3700 samplers
overflow	nutating disk meter	"in-line sampler"
under flow	nutating disk meter	"in-line sampler"

For locations where using automated samplers proved to be ineffective, an in-line sampler was devised to obtain flow proportionate samples. The in-line sampler conveys the flow through a one-inch diameter pipe and across an irrigation drip valve. Vinyl tubing attached to the drip valve conveys a small proportion of the total flow to a plastic bucket. The angle of the drip valve controls the amount of

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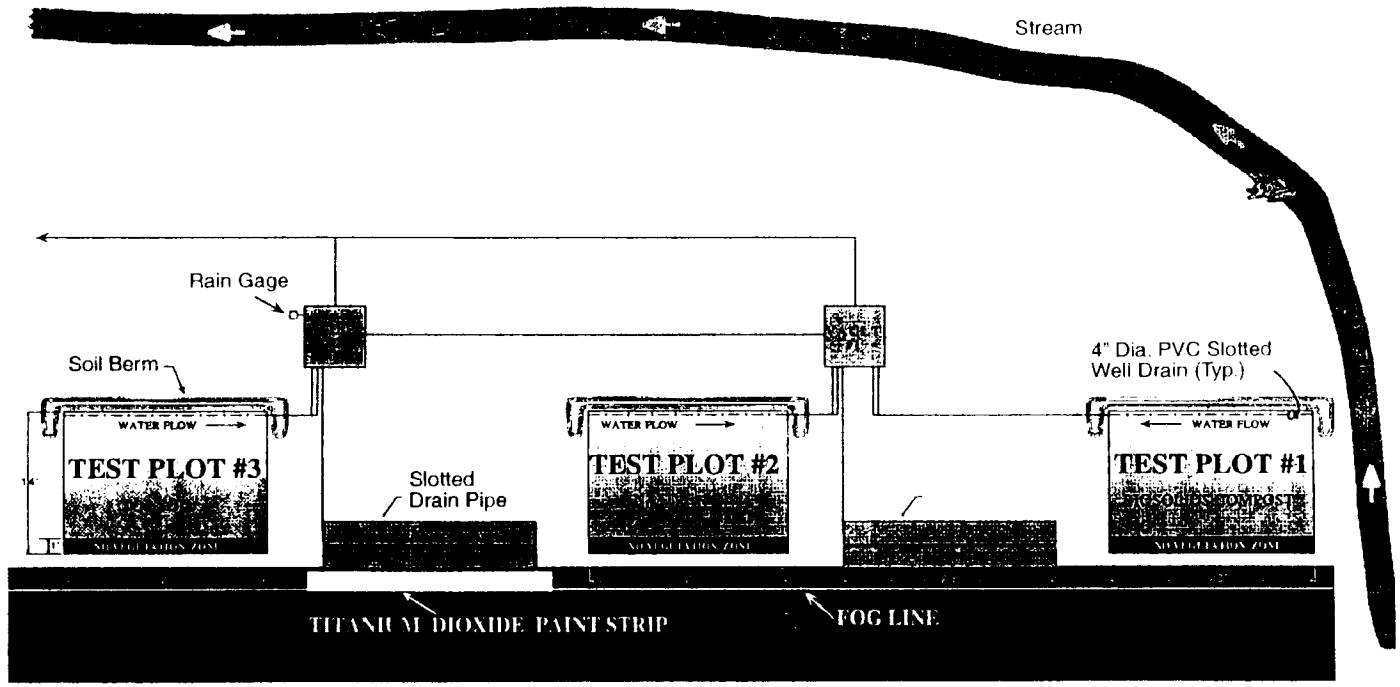

Figure 1. Filter strip BMP plan view site layout

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flow. The vinyl tubing can also be pinched to further reduce the flow.

Ecology Embankment

Research Objective

The research objective is to gather the needed data to see if the ecology embankment can be accepted as a standard BMP in our HRM. The performance of the ecology embankment will be compared to that of the bioswale.

Design

The ecology embankment design was developed as part of a highway improvement project in King County. The project had limited space to install a standard BMP due to wetlands, streams, and riparian buffer zones. The ecology embankment is a modification of the bioswale. The ecology embankments will provide water quality treatment of highway stormwater runoff using the space available in the side slopes of the highway prism to filter out solids suspended in highway stormwater runoff.

Figure 2 shows a cross section of the ecology embankment. The design contains an 8-inch PVC underdrain pipe in a 2-foot-wide trench bedded with gravel at its base. The underdrain pipe and trench allows conveyance of treated runoff.

Above the pipe trench, the embankment contains a minimum 1-foot layer of a mixture of soil and soil amendments, the ecology mix. The ecology mix layer is overlain by a porous geotextile mat. The mat is crush-resistant, pliable, resilient, water-permeable, and highly resistant to chemicals and decomposition. The exposed surface of the embankment is seeded, fertilized, and mulched twice.

A slot drain collects control samples for untreated runoff water quality and runoff volumes. A sampling station vault houses monitoring equipment including 0.4 HS flumes, nutating disc meters, deep-cycle marine batteries, and other equipment.

Difficulties

Accurately recording the discharges through the system may be difficult. We anticipate that the monitoring techniques we ultimately selected for the filter strip research site will work here as well.

The maintenance office was very concerned with the ecology mix and drain pipe associated with this BMP. First, maintenance states that the ecology mix does not have the structural integrity to support vehicle loads. They fear that the travelling public may drive off the road and get stuck in the embankment. Secondly, because the drain pipe requires occasional flushing, the maintenance office felt

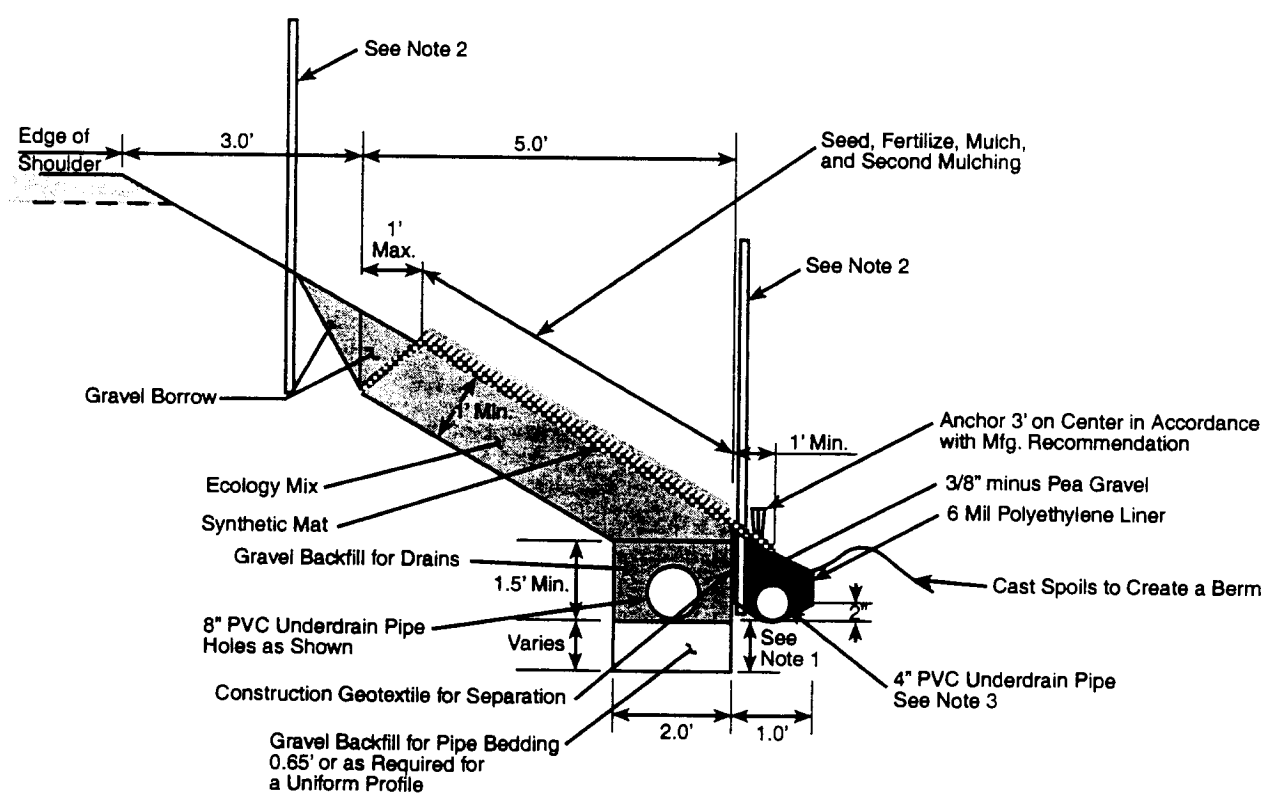

Figure 2. Modified ecology embankment cross section A-A.

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that the ecology mix wouldnt support their heavy equipment. The design was modified to allow for flushing of the drain pipe from the edge of pavement. Clean outs will be constructed on the roadway shoulder at a considerably increased cost.

Swirl Concentrator Systems

Research Objective

The Stormceptor™ is a commercially available vault system that utilizes swirl concentrator technology. The primary objective of monitoring the Stormceptor™ is to verify pollutant removal rates independent of manufacturer claims. The manufacturer reports solids removal rates at greater than 85% for low-flow events, with significantly less removal capacity at higher-flow events. Since the manufacturer emphasizes oil and grease removal as one of its main features, grab samples will be collected to provide better estimates of floatable hydrocarbon removal. The second objective is to collect maintenance requirements data.

Design

The Stormceptor™ was selected as part of a highway improvement project in King County. Since the project had limited space to install a standard BMP due to buildings and Metro bus stops, the Stormceptor™ units are installed beneath the pavement in lieu of a catch basin or manhole.

Figure 3 shows the Stormceptor™. It is a dual-level vault designed for ultra-urban settings to enhance the removal of sediments and oil. The Stormceptor™ is divided into a lower storage/separation chamber and upper bypass chamber.

Normal flows are diverted into the lower treatment chamber where oil and other light non-aqueous phase liquids rise. They then become trapped and suspended solids settle to the bottom of the chamber by gravity and centrifugal forces. During high-flow conditions, the bypass chamber conveys water to the downstream storm sewer directly circumventing the lower chamber. This prevents the resuspension and scour of settled pollutants.

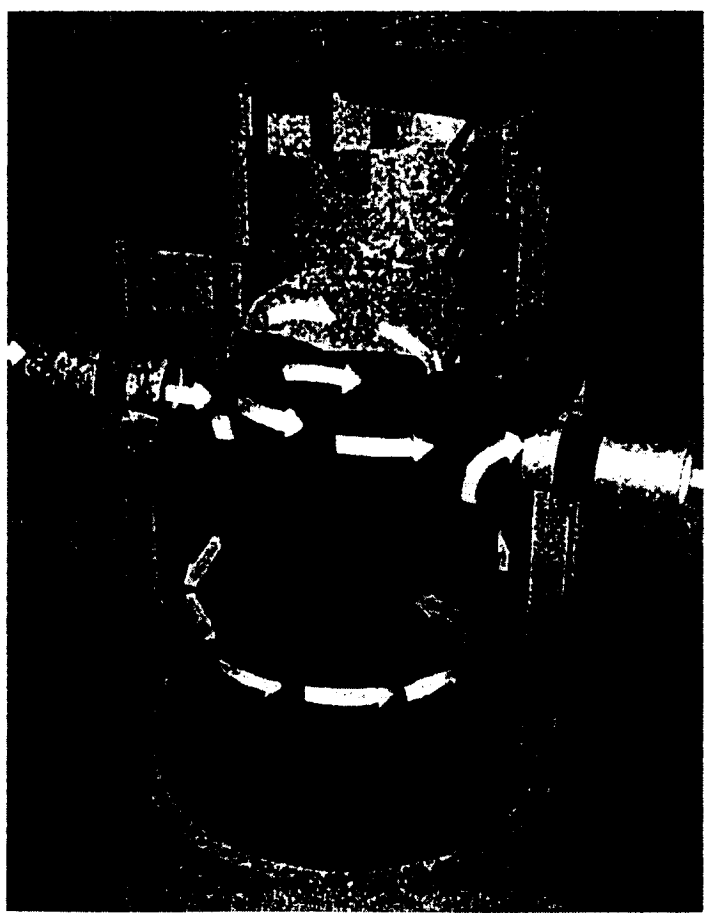

Figure 3. Stormceptor™.

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Difficulties

Safety and traffic were major concerns. Placement of the Stormceptor™ vaults along an urban, unlimited access highway, coupled with a dedicated High Occupancy Vehicle (HOV) lanes with Metro bus stations made selection and placement of monitoring equipment difficult. In addition, change orders during the construction phase forced us to step back and re-evaluate our sampling plan. The change orders made it impracticable to use automatic sampling equipment. It was decided that collection of time-composted grab samples was the only option. Grab samples will be collected at the catch basin preceding the BMP and at the outfall to an urban creek.

Enhanced Wet Detention Ponds

Research Objective

WSDOT would like our existing wet ponds to work better. In areas, where native soils have a high clay content, stormwater runoff from construction sites remains turbid even after being detained and passed through a wet pond. The discharges often exceed water quality standards for turbidity and suspended solids. WSDOT is investigating the use of coagulants to improve turbidity removal.

A group of long-chain polymers called polyacrylamides (PAM) were selected to test. Only the anionic form of PAM will be tested. Nonionic and cationic forms of PAM will not be used in WSDOT's experimental BMP because of toxicity considerations. Two PAM products which will be tested in WSDOT's innovative coagulation/flocculation BMP - Cytek Industry's Magnifloc™ 866 A and Magnifloc™ 905N flocculants.

WSDOT plans to test PAM at a detention pond in southern King County. This area has predominately clay loam glacial outwash soils. These soils have a history of problems with mass wasting, erosion control, and water quality because of high levels of turbidity. Slope stability problems preclude construction of additional BMPs. The site receives runoff from approximately 15 acres, 4 of which are currently paved. Monitoring of grab samples of pond effluent registered turbidity readings which, on occasion, have exceeded 200 NTUs.

PAM will be tested at a second pond along SR 5 - Leverich Park near Vancouver, Washington. The detention pond which receives SR 5 runoff was constructed in 1978. During large precipitation events, high levels of turbidity have been observed in the Leverich Park detention pond because of influent highway runoff and resuspension of sediments residing in the pond. PAM can enhance the effectiveness of this detention pond by flocculating suspended sediments and preventing resuspension of sediments that may get discharged to Burnt Bridge Creek.

Design

No special construction provisions are needed for using PAM to enhance sediment removal in stormwater detention ponds. Standard construction and design practices for stormwater detention ponds are adequate.

PAM delivery system will be a "tea bag" device. This can hold granular PAM and be anchored within the influent pipe or channel. During precipitation events, stormwater is expected to infiltrate through the granular PAM tea bag. The PAM mixed with the sediment-laden stormwater is expected to flow into the detention pond, where velocities would be reduced and the flocculation would result.

Preliminary Results

Preliminary results are very promising. Jar tests using sediment laden construction site runoff demonstrate greater than 95% turbidity removal in 30 minutes with 2ppm (parts per million) PAM dosage. The water turned clear, looking nearly like drinking water and the sediment was captured in fluffy, quick settling flocs. The jar tests also demonstrate that the flocs form at as little as 0.8 ppm PAM dosage. Aluminum chlorohydrate is an efficient coagulant and acts as a bridging agent between solids and PAM. Jar tests demonstrate that a one-to-one ratio of aluminum chlorohydrate and PAM further improves the settling process.

Difficulties

WSDOT has been unable to initiate full-scale field tests using PAM. The Department of Ecology is very concerned about the addition of chemicals to the waters of the state. Standard testing protocols are not available and have delayed the testing of this potentially very effective product. Specifically, short-term and long-term aquatic toxicity testing are required.

Ship Canal BMP Research Facility

Research Objective

The objective is to build a full-scale ultra urban highway runoff research facility. The facility will be the testing grounds for new stormwater BMPs designed for limited space situations. Our goal is to have sufficient performance data to be able to select and install BMPs at the lowest possible costs that comply with water quality standards.

Design

WSDOTs most ambitious project is the Ship Canal Stormwater BMP Facility, located in Seattle. This research facility will simultaneously evaluate up to six experimental BMPs treating stormwater runoff from WSDOTs busiest freeway. The use of controls will minimize bias and improve objective comparative evaluations of BMP performance and maintenance requirements. Construction is currently scheduled for the Summer of 1998.

Figure 4 is a schematic of the Ship Canal Research facility. The Research facility will contain four test bays. Space is allowed to add two additional test bays. Commercially available BMPs or custom-designed BMPs can be evaluated at the site.

Difficulties

A local community action group was concerned about polluted discharges from the facility, in the event that one

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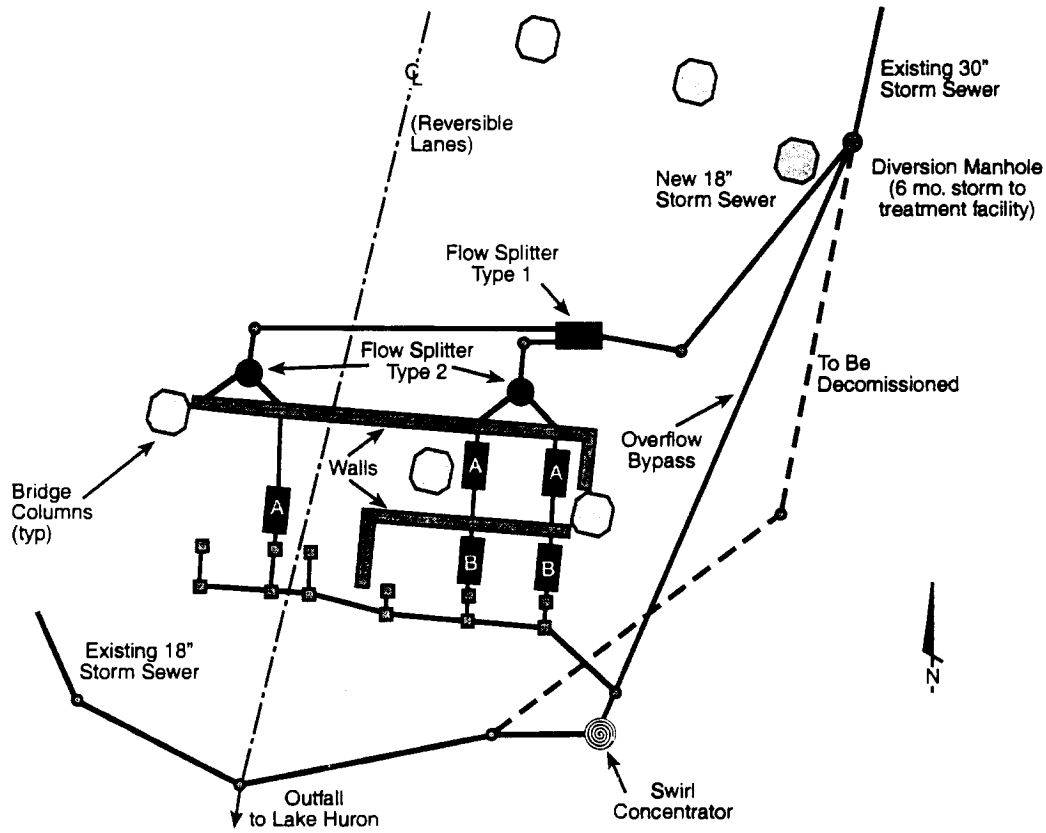

Figure 4. Schematic of ship canal.

of the units failed or had poor performance. Therefore, the design calls for a swirl concentrator-type BMP as a polishing step before discharge into Lake Union.

Because the facility will be built in an urban location, the designer had to work around many issues. Water, cable, and fiber optic lines will need to be relocated. The facility also had to work around the bridge columns. Walls will be constructed because of the large changes in elevation.

The design was first completed in November 1997. The bids were 42% higher than anticipated. Sufficient funds were not available to construct the facility. A second, scaled-back design will be completed during the month of February 1998. We plan to go out to bid during the month of March.

Challenges

There are many challenges when conducting innovative BMP performance monitoring. Finding good research sites takes time and requires thorough research. Major recommendations are:

- Select a safe site for staff and for the traveling public. Give consideration to highway traffic patterns and access. Can sampling staff exit and enter the highway SAFELY?
- Know the hydraulics of the site. In the field there is no such thing as sheet flow. Dips, gouges, bumps and irregularities direct and redirect water into or around the test site. Flow patterns during various storms should be factored. Sites that include drainage from off-site contributors should be eliminated to avoid having to sample all discharges into the site.
- Minimize or eliminate confined spaces from the design. Confined space regulations limit the vertical depth of our sampling vaults to a maximum of four feet; however, the vaults should be deeper to allow for the vertical distance needed for water quality monitoring. Our most recent vault designs allow for both by utilizing a false floor that the sampling staff can safely stand on. Sections of the floor can be removed as needed.
- Minimize travel time. WSDOT tries to locate research facilities within 15 to 20 minutes of the office and within an hour of an environmental laboratory. The longer the commute, the more difficult it is to collect or deliver samples and to pick up spare parts and to make the repairs necessary to keep the facility operational.
- Supervise the construction: as drawn does not mean as built. Review the plans with the project engineer. Then check and confirm the construction as shown in

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the plans. Stay in touch and make sure that all change orders are approved by the research staff.

- Allow time to listen and understand resource agency concerns. Balance the need for more effective BMPs against the need for complete environmental understanding and the fate and function of a BMP. If chemicals are added to the water, include short- and long-term toxicity testing and hazardous waste testing of sludges as an element of the testing protocols.

- Innovation means change. Change threatens many people and upsets traditional organizational roles and responsibilities. The next challenging step is to incorporate the research results into how highways are designed and to have the regulating agencies accept designs.

Conclusions

The Washington State Department of Transportation has earned a reputation as an innovative developer of large public works projects. Constructing and maintaining transportation facilities will always be the primary mission of WSDOT.

However, the agency accepts responsibility for the potential environmental and social impacts of our facilities. A major responsibility is to protect our state's water quality and to preserve our environmental values. WSDOT has adopted the watershed-need philosophy as the best way to provide cost-effective water quality treatment.

The Innovative BMP Research program will help provide clear selection criteria and design parameters for stormwater BMPs. This, in turn, will provide the best BMP for each project or outfall site. As the years go by, WSDOT anticipates a diminishing impact to watersheds from its transportation facilities.

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StormTreat™ Technology for Stormwater Treatment

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Stormwater treatment standards throughout the nation have focused on removal of suspended solids, trusting that other contaminants will be removed as well. Massachusetts, for example, requires 80% suspended solids removal. While this is an important first step, there are a wide range of other contaminants such as fecal coliforms, nutrients, metals and hydrocarbons that may not be treated by technologies focusing solely on suspended solids removal.

The StormTreat™ System (STS) was developed in 1994 in response to the need to provide enhanced treatment of stormwater, beyond suspended solids removal. The technology is designed to capture the "first flush" of runoff, and provide treatment in a 9.5-foot diameter tank through sedimentation, filtration and constructed wetlands uptake. Two years of independent testing results indicate removal rates of 97% for fecal coliform bacteria, 90% for phosphorus, 77% for dissolved nitrogen and 99% for total suspended solids.

StormTreat™ is the first stormwater technology to be verified by the Massachusetts Strategic Environmental Partnership (STEP) Program, a state program designed to verify the claims and effectiveness of new technologies. The Massachusetts STEP program is part of a six-state Partnership for Environmental Technology, including Illinois, California, Pennsylvania, New Jersey and New York.

StormTreat™ has recently developed an optional infiltration feature to treat stormwater from as much as one impervious acre per unit. This is accomplished by infiltrating treated water directly into surrounding and underlying soils. With this option, treatment costs can be reduced to as low as $7,000 per acre treated (including installation).

Introduction

Stormwater runoff from streets, parking lots and adjacent areas is one of the most significant water pollution problems today. Lakes, reservoirs, streams, coastal waters and related wetlands receive "pulses" of oils, metals, bacteria, nutrients and other pollutants during and following each storm event. Where stormwater is infiltrated or injected into groundwater, impacts may occur to subsurface drinking water supplies.

Chronic petroleum hydrocarbon discharges to the marine environment from stormwater runoff far outweigh those from catastrophic oil spills from tanker ships (such as the Exxon Valdez). Researchers associated with the EPA-sponsored Buzzards Bay National Estuary Program have concluded that stormwater-derived bacterial loadings are responsible for the majority of shellfish area closures. Stormwater has also been documented as a major cause of eutrophication of lakes and ponds nationwide.

Non-point sources of pollution such as stormwater typically originate from diffuse areas. Stormwater is generated from streets, parking lots, roof tops, driveways, lawns, agricultural fields and forests (Figure 1). Frequently, stormwater runoff from several of these various "land uses" is combined into a stormwater flow in a drainage ditch or stormwater pipe which ultimately discharges to a receiving water where the impacts are realized. Because of the diffuse nature of sources, stormwater management is best accomplished by a watershed management technique which treats each area within the watershed independently as opposed to the more conventional "big pipe" solution where a large detention/treatment system is constructed at the bottom of a watershed attempting to catch and treat all of the stormwater generated within the watershed.

There is a direct relationship between the traffic volume on a road and the concentrations of metals found in the stormwater. This suggests that stormwater quality is likely to degrade over time as urbanization continues and land use densities increase. A study conducted by Arnold and Gibbons (1996) has shown a direct relationship between the percentage of impervious cover and the quality of adjacent surface waters. When impervious surfaces cover 30% of the land area, significant water quality impairment is evident.

While there is a wide variety of compounds present in stormwater, the majority of treatment approaches focus

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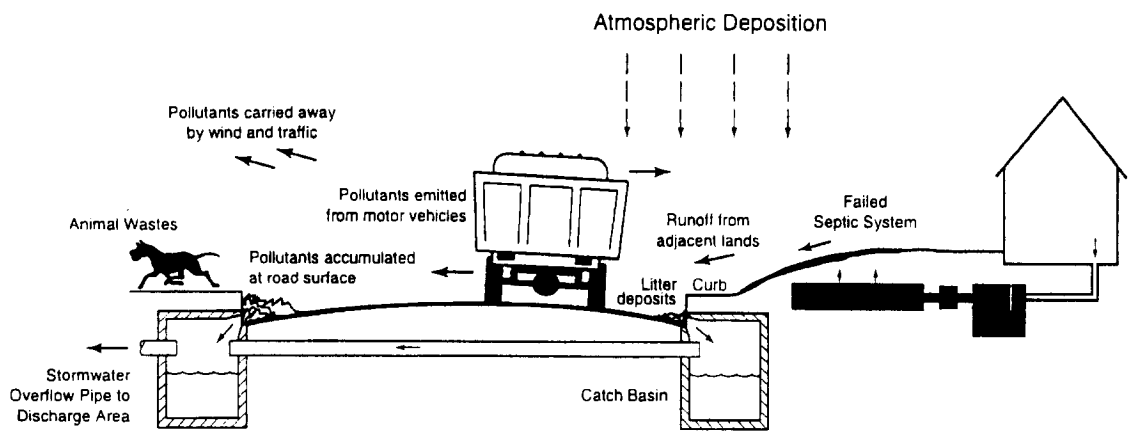

Figure 1. Stormwater Pollutant Pathways.

on removing suspended solids. The premise is that if suspended solids are removed, a wide range of other pollutants associated with the solids are removed as well. Removal of suspended solids is used as a performance standard in numerous stormwater regulations. The most recent example may be the State of Massachusetts which now requires 80% suspended solids removal for treatment systems approved within 100 feet of inland or coastal wetlands (MA DEP, 1997).

There are two problems with this thinking. First, the majority of the suspended solid particles (Table 1) are very small, with 78% of them less than 44 μm in diameter (Rexnord, 1984). This presents significant problems with respect to best management practices (BMPs), which rely wholly upon physical settling, or separation, of solids. The settling rates associated with silt and clay-sized particles are on the order of several hours to days per foot of settling. This has significant implications with regard to the effectiveness of detention basins and other treatment approaches in meeting performance standards.

Second, a significant percentage of most pollutants is associated with the finer particles (Table 2). For example, approximately 56% of phosphates in stormwater is associated with particles smaller than 43 μm. If the smaller particle sizes are not removed, the expected treatment of other constituents is not obtained.

Table 1. Wet Sieve Analysis of Highway Runoff Composite Samples

	Percent of Suspended Solids				
Particle Size (μm)	Sacramento Hwy. 50	Harrisburg I-81	Milwaukee I-94	Effland I-85	Mean
> 250	1.54	6.10	14.56	3.58	6.45
88-250	9.07	6.70	7.00	1.30	6.02
44-88	10.70	11.70	5.84	8.06	9.08
< 44	78.69	75.50	72.60	87.06	78.45

Source: Rexnord, Inc., 1984

Generally speaking, a "first flush" effect is observed with stormwater quality. The highest pollutant concentrations are typically observed at the beginning of a storm event. This is because of the residues which are available on paved surfaces at the beginning of a rain event. As this first flush is washed from the paved surface, pollutant concentrations typically decline throughout the remainder to the storm event. The "first flush" principle has been commonly observed for total suspended solids. Up to 90% of the total suspended solids (TSS) are contained within the first 0.5 inches of runoff (EPA, 1974).

There are exceptions to the first flush principle. For example, a long light rain followed by a strong downpour may exhibit its highest concentrations of pollutants toward the end of a storm event. However, when one averages all rainstorms, the majority of annual pollutant loading occurs during the first flush. Another exception is a large watershed which has a long "time of concentration" (the time required for water to flow from the uppermost part of the watershed to the final point of discharge). In these cases, samples taken at the discharge point may integrate the first flush from the bottom of the watershed but miss the first pollutant loadings from the upper part of the watershed.

The StormTreat™ System

The StormTreat™ System was designed in 1994 in an effort to treat stormwater in a way that would effectively remove a broad range of pollutants in addition to suspended solids. It saves space by reducing the need for unsightly detention basins. It captures and treats the first flush of runoff which contains 90% of pollutants. An optional infiltration feature provides for the treatment of larger quantities of stormwater, beyond the first flush, as described later in this paper.

The system consists of a series of six sedimentation chambers and a constructed wetland which are contained

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Table 2. Percent of Street Pollutants in Various Particle-Size Ranges

	Particle Size (μm)					
Pollutant	>2000	840-2000	240-840	104-240	43-104	<43
Total solids	24.4	7.6	24.6	27.8	9.7	5.9
Volatile solids	11.0	17.4	12.0	16.1	17.9	25.6
COD	2.4	4.5	13.0	12.4	45.0	22.7
BOD_5	7.4	21.1	15.7	15.2	17.3	24.3
Phosphates	0	0.9	6.9	6.4	29.6	56.2
All toxic metals	16.3	17.5	14.9	23.5	—	27.5
TKN	9.9	11.6	20.0	20.2	19.6	18.7
All pesticides		27.0			73.0	
PCBs		66.0			34.0	

Source: EPA, 1983

within a modular 9.5-foot diameter tank (Table 3). It is constructed of recycled polyethylene which connects directly to existing drainage structures.

Influent is piped into the sedimentation chambers where larger-diameter solids are removed (Figure 2). The internal sedimentation chambers contain a series of skimmers which selectively decant the upper portions of the stormwater in the sedimentation basins, leaving behind the more turbid lower waters. The skimmers significantly increase the separation of solids compared with conventional settling/detention basins. An inverted elbow trap serves to collect floatables such as oils within one chamber of the inner tank. After moving through the internal chambers, the partially treated stormwater passes into the surrounding constructed wetland through a series of slotted PVC pipes.

The wetland is comprised of a gravel substrate planted with bulrushes and other wetland plants. Unlike most wetlands constructed for stormwater treatment, StormTreat™ conveys stormwater into the subsurface of the wetland and through the root zone, where greater pollutant attenuation occurs through such processes as filtration, adsorption, and biochemical reactions.

Precipitation of metals and phosphorus occurs within the wetland substrate while biochemical reactions, including microbial decomposition, provide treatment of the stormwater prior to discharge through the outlet valve. An outlet control valve provides up to a 5-day holding time within the system. The valve can be closed to contain a hazardous waste spill.

Table 3. StormTreat™ Specifications

StormTreat™ Specifications	
Diameter	9.5 ft
Height	4 ft
Storage Capacity	1390 gal
Inflow Pipe Diameter	4 in
Outflow Pipe Diameter	2 in
Average Detention Time	5 days
Average Discharge Rate	0.25 gal/min
Tanks Required per Acre of Impervious Surface	2-5 Tanks*

* The number of tanks depends upon the level of treatment required, the in-line detention capacity and the use of the optional infiltration feature.

The size and modular configuration of StormTreat™, makes it adaptable to a wide range of site constraints and watershed sizes. It can be installed in any type of soil as the discharge rate is very low (0.25 gal/min) and the gravel filter/wetland substrate is contained within the system. As the flow through the system is gravity dependent, the system requires an elevation change from the pavement surface to a discharge point of at least 4 feet.

StormTreat™ has been installed in a variety of applications, including commercial parking lots, industrial sites, town landings and marinas, transportation facilities and residential subdivisions. It is an appropriate treatment technology for both coastal and inland areas, and can be used throughout the country with only minor system modifications to fit local conditions.

To date, 315 analyses have been conducted on 33 samples which have been collected over eight independent storm events during both winter and summer conditions in New England. Influent stormwater samples were taken at the entry point to the StormTreat™ tanks at the catch basin. Effluent samples were taken during the 5 days following the storm event. The quality of the sampled effluent was then compared with influent and removal rates were computed. Test results are summarized in Table 4.

StormTreat™ requires minimal maintenance. Annual inspection is recommended to ensure that the system is operating effectively. At that time the manhole is opened and the burlap grit screening bag covering the influent line should be removed and replaced; filters should be removed, cleaned, and reinstalled. Sediment should be removed from the system via suction pump once every 3-5 years, depending on local soil characteristics and catch basin maintenance practices.

State Verification

StormTreat™ is the first and only stormwater treatment technology to be verified by the Massachusetts STEP Program. The Strategic Environmental Partnership (STEP) Program is a service provided by the Commonwealth of Massachusetts to help develop new environmental technologies and to verify their effectiveness. Both business management and technological expertise is provided through the University of Massachusetts. An excerpt from the executive summary of this assessment reads as follows:

"It is the conclusion of this assessment that the (StormTreat™) system, when sized according to recommended criteria, with proper operation and maintenance, can provide levels of treatment required under Standards 4 and 6, as specified by the Massachusetts DEP Stormwater Management Handbook. Under special circumstances, the system may provide as much as 98% removal of TSS when sized according to design criteria. The system, when configured for recharge can meet Stan-

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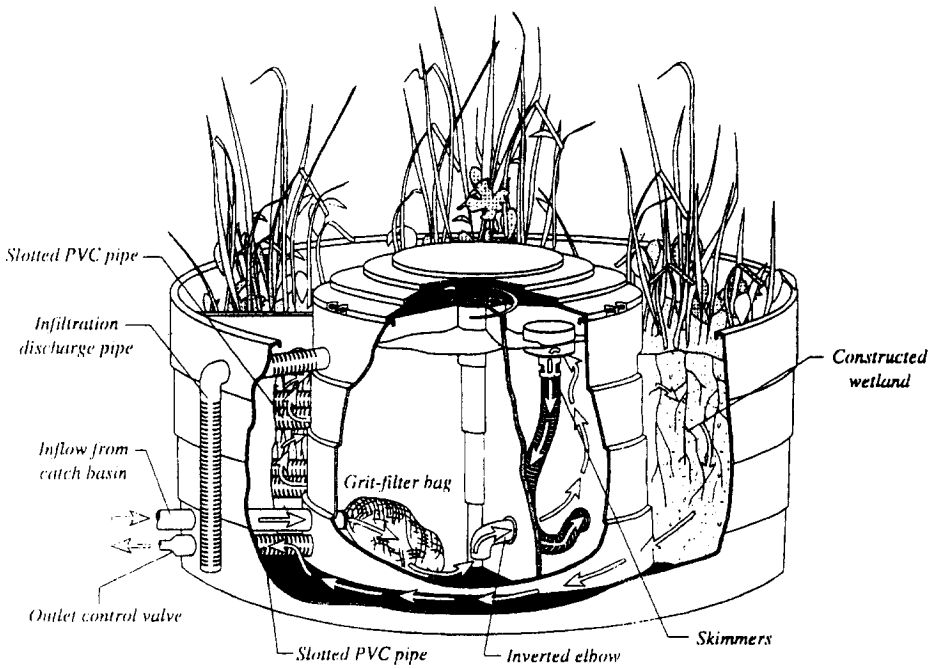

Figure 2. Schematic cross section of StormTreat™ system.

Table 4. Water Quality Sampling Results, Kingston, MA

Pollutant	Average Stormwater Influent	Average Treated Discharge	Percentage Removed (%)
Fecal coliform (no./100 ml)	690	20	97
Total suspended solids (mg/l)	93	1.3	99
Chemical oxygen demand (mg/l)	95	17	82
Total dissolved N (mg/l)	3569	520	77
Total Petroleum Hydrocarbons (mg/l)	3.4	0.34	90
Lead (mg/l)	6.5	1.5	77
Chromium (mg/l)	60	1	98
Phosphorus (mg/l)	300	26.5	90
Zinc (mg/l)	590	58	90

Note: Samples were collected by the Jones River Watershed Association in accordance with EPA sampling protocol, and analyzed at state-certified laboratories (Schueler, T. 1995).

dard 3 and is also likely to meet Standard 5, for land uses with higher potential pollutant loads, when sized according to design criteria."

The Massachusetts STEP is a member of the Six State Partnership for Environmental Technology which includes California, Pennsylvania, Illinois, New Jersey, and New York. Reciprocal certifications in these states may be available.

Infiltration of Treated Stormwater

An optional infiltration feature enables the StormTreat™ System to process as much as one acre of impervious surface area per unit. This is accomplished by directing *treated* stormwater into the surrounding and underlying soils (Figure 3). There are several advantages to this approach. First is the replenishment of groundwater. Under natural (undeveloped conditions) a certain percentage of precipitation infiltrates through the soils and recharges underlying groundwater (this may be as high as 50% of the annual precipitation resulting in groundwater recharge in the Northeast US). Commonly, the development of land results in an increase of impervious surfaces reducing the recharge rate (in some cases to zero). Less recharge means less water supply availability, declining water tables and less baseflow (discharge) to nearby streams and wetlands.

A second advantage of this new feature is significantly-increased treatment and lower costs to customers (as low as $7,000/acre installed). By allowing treated stormwater to infiltrate into the surrounding soils, additional treatment capacity is achieved. This is accomplished first by infiltrat-

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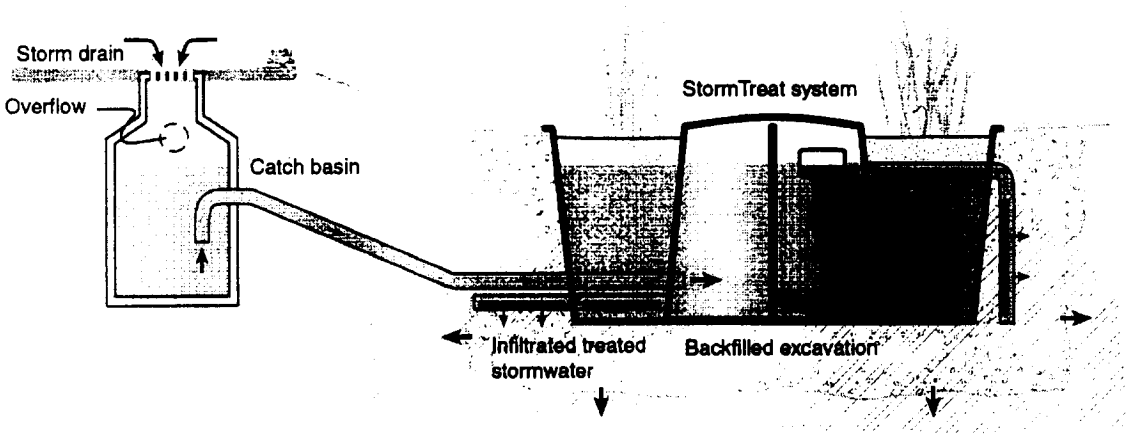

Figure 3. Schematic cross section showing new infiltration feature.

ing into the surrounding backfill materials (specified as 3/4-inch stone). This stone is highly permeable and serves to transmit the treated water downward and laterally until it encounters the parent soils. During peak flow periods, the infiltration rate may exceed the permeability of the parent soils and the stone backfill area serves as a temporary storage reservoir.

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Evaluation of Stormceptor® and Multi-Chamber Treatment Train as Urban Retrofit Strategies

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Introduction

The installation of water quality best management practices (BMPs) in developed urban areas is problematic. A landscape comprised of buildings and pavement presents little opportunity for placement of new BMPs. To overcome this obstacle of limited space, a new set of retrofit BMP technologies are emerging which utilize space underground, thereby avoiding disruption to current aboveground land uses. This paper evaluates the water quality benefits of two retrofit BMPs, the Stormceptor and the Multi-Chambered Treatment Train (MCTT). The installation of these devices and the subsequent evaluations were cooperative efforts involving the US Environmental Protection Agency, US Geological Survey, cities of Milwaukee and Madison, (Wisconsin), StormceptorÆ Corp., University of Alabama-Birmingham, and Wisconsin Department of Natural Resources.

Study Design

Description of Test BMPs

The Stormceptor consists of a treatment tank and bypass chamber (Figure 1). Water initially enters the bypass chamber from an upstream stormsewer. During periods when the flow doesn't exceed the unit's capacity, water is diverted down a drop pipe, where water is discharged tangentially along the chamber's wall. Suspended sediment falls to the bottom of the tank where after a period of accumulation, it is pumped out and landfilled. Water exits the treatment chamber at the opposite end through a similar drop pipe and drains to the downstream stormsewer. Hydrocarbons and other lighter-than-water materials are trapped above the treatment tank's drop pipes. During periods of surcharge, the portion of the stormwater in excess of the treatment rate flows directly over the weir in the bypass chamber and receives no treatment. The unit chosen for this project was the STC 6000. Its capacity is 6150 gallons and it is designed for a treatment flow capacity of 800 gal/min. The treatment tank is 10 ft in diameter and 10 ft deep.

The MCTT consists of three components, an inlet area, a settling chamber, and a filter bed (Figure 2). The largest grit material accumulates in the bottom of the 4 ft inlet basin. In addition, water passes over a mesh bag of column packing balls which enhance aeration and loss of highly volatile components. The second chamber has inclined tube settlers which further enhance the settling process. This chamber also contains sorbent pads which remove floatable hydrocarbons. Water drains slowly from the second chamber into the filter bed chamber via a 0.35-inch orifice. This final chamber contains a mixed media of sand/ peat/ activated carbon supported by filter fabric and is designed to remove fine particles along with some dissolved constituents via sorption and ion exchange. The second and third chambers are constructed from a partitioned concrete box (10 ft wide x 19 ft long x 5 ft high). The capacity of the settling chamber is 750 ft^3 although the height of the orifice results in a "dead storage" capacity of 375 ft^3, leaving the actual storm volume capacity at 375 ft^3. Once this capacity is reached, the excess water is bypassed.

Site Descriptions

Both study sites are public works maintenance yards used for fueling, storage and maintenance of city vehicles, most of which are heavy trucks. In addition, open storage of sand, salt and yard wastes can be found at times. The Stormceptor was installed at the Badger Rd. public works garage in Madison, Wisconsin. One stormsewer inlet collects the runoff water from the entire facility (4.3 acres) and the Stormceptor was retrofitted in the existing stormsewer approximately 300 ft from the inlet. The MCTT was constructed at the Ruby St. garage in Milwaukee, Wisconsin. The unit receives water from only a portion (0.2 acres) of the paved area.

Sampling Design

Similar sampling strategies were employed for the evaluation of both these devices. The evaluation consisted basically of collecting flow-integrated samples at the influent and effluent for each BMP. In addition, the Stormceptor's

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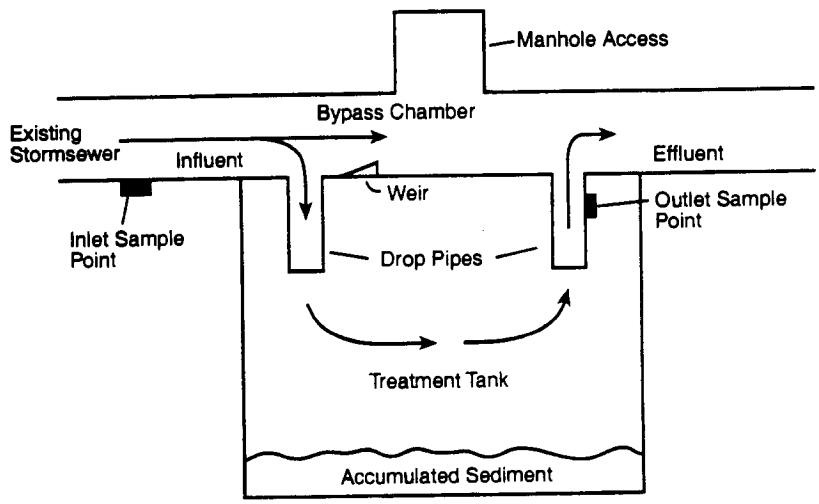

Figure 1. Cross section of Stormceptor® device showing placement of sampling points.

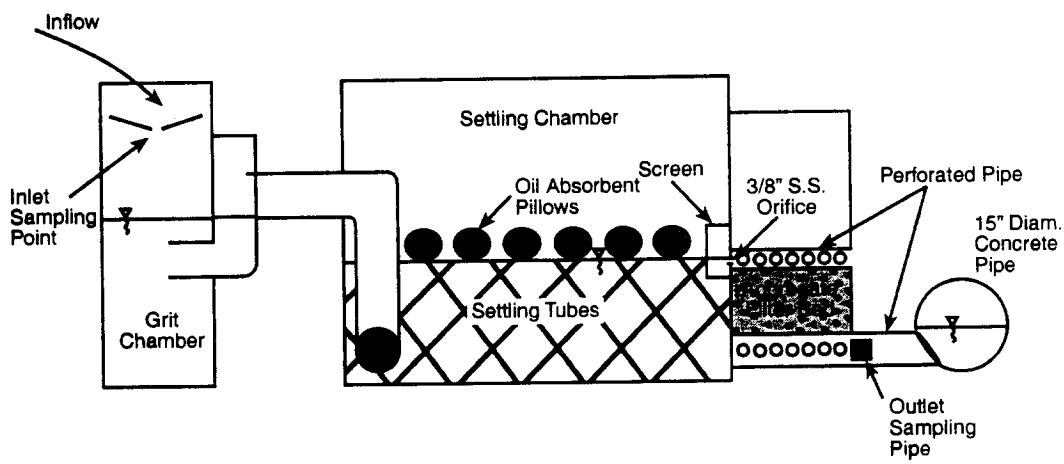

Figure 2. Cross section of MCTT device showing placement of sampling points.

bypass water was also collected. The locations of sampling ports are shown in Figures 1 and 2.

Sampling equipment and monitoring instrumentation were housed in small onsite buildings. In addition, tipping bucket rain gages were placed on nearby rooftops. Dataloggers served as the site controller, with a modem and telephone for external communications. At both sites, flow was measured either directly, with a doppler or electromagnetic flow meter, or indirectly, with a pressure transducer/stage height measurement. The datalogger was programmed to initiate rainfall, stage, and velocity measurements on a variable time scale. Measurements were taken more frequently during runoff periods and less frequently during dry periods. The data were recorded using internal memory of the datalogger and a backup storage module, and transferred every 24 hours via modem to the USGS computer in Madison.

Both sites had two refrigerated samplers equipped with peristaltic pumps and Teflon-lined sample tubing to collect water quality samples. The automatic samplers were triggered by the datalogger to initiate collection of storm samples. One composite sample each was collected at the inlet and the outlet for each storm. Each sample contained between 5 and 40-stormflow volume-weighted subsamples, which resulted in event-mean concentrations.

Fifteen consecutive storms were monitored at the MCTT site from April 29, 1996 through September 8, 1996. A total of 68 constituents were measured in the samples taken at the MCTT device. At the Stormceptor site, 45 storms were monitored from the period of August 6, 1996 to May 1, 1997. Samples from 15 of the 45 storms were analyzed for 37 constituents included a variety of solids, nutrient, metals and polycyclic aromatic hydrocarbons (PAHs). The remaining 30 storm samples were analyzed only for total

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suspended solids (TSS), total dissolved solids (TDS) and total phosphorus (TP). In both studies, loads and removal efficiencies were calculated (load=storm volume x event-mean concentration). Two types of removal efficiencies were calculated: tank efficiencies based on reduction of load of stormwater passing through the tank, and overall removal efficiency, which also accounts for load which is bypassed. In addition, particle-size characterization was performed on 15 samples at each site. Only a few constituents are presented here to highlight the studies' findings. Complete results can be obtained from the USGS Water Resource Division in Madison, Wisconsin.

Results

Stormceptor

Precipitation for the 45 monitored storms ranged from 0.02 to 1.31 inches. This rainfall produced runoff amounts ranging from 120 ft^3 to 30,000 ft^3. Though for 24% (11 out of 45 storms) water bypassed the unit, the total water volume that bypassed equaled only 9%. Stormwater was observed bypassing the treatment tank at flows greater than 500 gal/min, which is less than the manufacturer's specification of 800 gal/min. This difference may have been caused by the exit sewer pipe being slightly higher in elevation than specifications called for, causing a back pressure through the unit and resulting in the unit bypassing more often.

The influent total suspended solids (TSS) concentrations found are comparable to parking lot and street runoff concentrations observed elsewhere in Wisconsin (Bannerman et al., 1993) and other locations (Ellis, 1986). The influent TSS event-mean concentrations for the 45 storms ranged from 43 to 1236 mg/l with a median value of 251 mg/l. In general, the highest influent TSS concentrations were observed in the winter months, presumably reflecting the high activity of sand/salt trucks in the yard area. The influent TSS load ranged from 0.45 to 224 kg. and the cumulative influent load for the 45 measured storms was 1670 kg. An estimated 91% of this load entered the treatment tank; the remaining 9% was bypassed.

The TSS concentrations of the water exiting the treatment tank had a median concentration of 151 mg/l and ranged from 45 to 615 mg/l, approximately half the range of the influent concentrations. The total load exiting the treatment tank (1044 kg.) resulted in an overall removal efficiency for the treatment tank of 26%. The effluent TSS load totaled 1294 kg. (from 4.3 acres) for the 45 storms, indicating an overall reduction efficiency (treatment tank + bypass) of 22%. Because of the hydraulic residence time of the treatment tank, effluent water can be a mixture of influent waters from a number of previous storms. Therefore, caution must be used in interpreting individual storm efficiency results. In general, the monitored storms had sufficient runoff water to replace the majority of water in the treatment tank. Eighty-four percent of the storms exceeded one tank volume, 62% exceeded two tank volumes. Figure 3 illustrates the relationship between the individual effluent load and overall efficiency. There appears to be considerable variation in efficiency for the smaller storms and, in general, as the storm loads become larger, the efficiency decreases.

Total Dissolved Solids (TDS) concentrations were also quite variable, ranging more than two orders of magnitude in both the influent and effluent (median influent and effluent concentrations were 3860 and 4700 mg/l, respectively). Similar to TSS concentrations, the TDS concentrations showed a marked increase during the winter season, presumably due to salt stockpiled onsite and spillage from trucks. The maximum TDS concentration (114,000 mg/l) was on 1/26/96. Negative removal efficiencies were found for both the treatment tank and the overall system (both at -19%). The fact that efficiencies were negative may be due to either measurement errors or possible dissolution of granular salt within the tank. It is interesting to note that

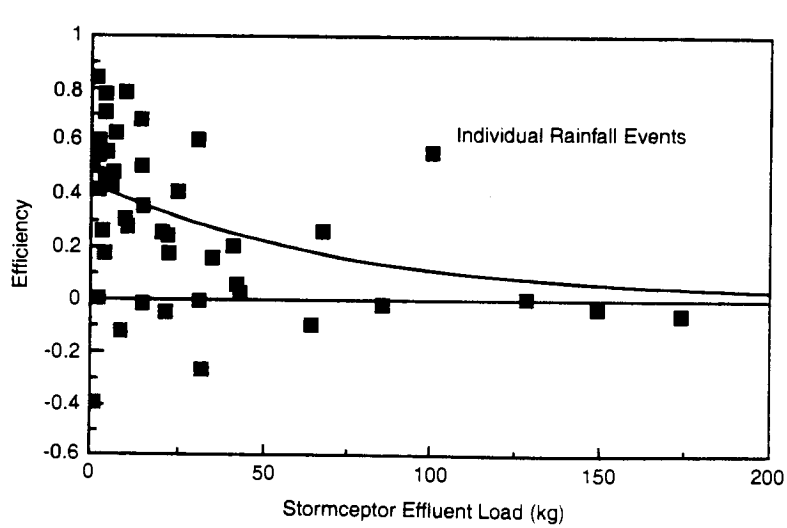

Figure 3. Relationship between individual effluent load and overall efficiency at the Stormcepter® site.

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the influent TDS load was more than 20 times the TSS load.

Total phosphorus (TP), the third constituent measured in all 45 monitored events had concentrations ranges from 0.08 to 1.3 mg/l in the influent and 0.06 to 0.86 in the effluent. The tank and overall removal efficiencies (20% and 18%, respectively) were somewhat less than TSS removal rates.

The remaining 34 constituents were measured in 15 storm samples only. In general, the removal efficiencies of the total constituent loads were similar to the TSS removal rates and none had negative efficiencies. Total polycyclic aromatic hydrocarbons exhibited the highest overall removal rate (32%). Of the four metals quantified, zinc had the highest concentrations and loading. The overall reduction efficiency for total zinc was 21%. Removal efficiencies of the dissolved constituents were always less than the total constituents with the exception of dissolved phosphorus, which interestingly was slightly greater than total phosphorus. Five dissolved constituents had negative efficiencies, which may of been a result of load errors, dissolution in the tank and redox processes in the tank's accumulated sediment. The increase in chloride mass further suggests that granular sodium chloride is going into solution after entering the tank. Given that the device is basically designed for particulate solids removal, the negligible removal of dissolved constituents was anticipated.

Particle-size analyses of the 15 influent and effluent samples showed little shift in the size distribution (Figure 4). The small clay-size fraction increased slightly from 3.3 to 3.6%. Silt was the predominant size fraction and increased from 88.9 to 93%; the sand fraction decreased from 7.8 to 3.5%.

At the end of the study period, the oily surface material, water, and bottom sediments were pumped from the treatment tank. Approximately 16 ft^3 of floating oily material was captured at the top of the treatment tank. As the tank was pumped down, the water was subsampled for TSS and TDS. An increasing gradient in TDS concentration was observed from top to bottom. At 7 ft from the top of the tank, the TDS had increased from 51,000 to 138,000 mgl. Greater water density caused by this high salinity could have hindered particulate solids settling. In addition, this sharp increase in density may have resulted in a portion (30-40%) of the tank's volume being resistant to mixing, thereby decreasing the treatment ability of the tank. Using the manufacturer's sizing guidelines, this decreased effective volume would cause a marked decrease in tank efficiency. But inconsistent with this hypothesis is the fact that the first 14 storms (before the saline buildup) exhibited a total removal efficiency of only 5% and the last 14 storms monitored (during the period of high saline conditions) had above average (25%) removal. Therefore, the impact of the salt on the tank's settling ability is unclear at this time.

The depth of sediment accumulated on the bottom was measured and then subsampled and analyzed for dry weight, particle size, metals and PAHs. As a mass balance check on the sediment material removed by the treatment tank, the measured mass of sediment deposited in the tank was compared to the difference in influent and effluent TSS mass. Based on sediment depth and dry weight, the mass was calculated to be 536 kg. This value compares favorably with a TSS load estimate of 473 kg. These similar mass values lend credence to the methodologies used to sample the stormwater flow and concentrations.

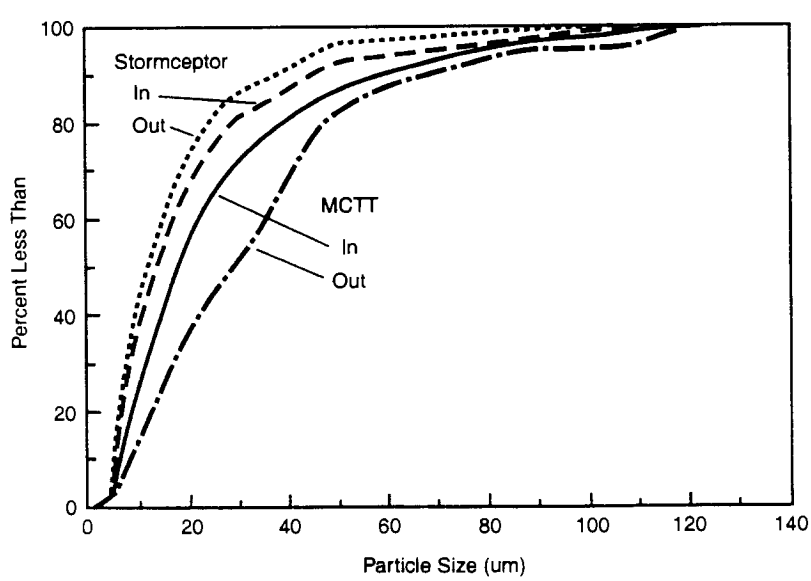

Figure 4. Mean particle-size distributions of influent and effluent TSS for both studies.

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The treatment tank was found to be quite selective with respect to removal of the larger-sized particles. The sieving of this material found 80% had a particle size of greater than 250 µm. This finding suggest the influent stormwater may have had a higher concentration of the sand-size fraction than was found in the suspended solids particle size analyses. Larger sand-size material, saltating along the bottom of the stormsewer pipe may have not been accounted for in the TSS loading. This unaccounted material would further explain why the TSS load (473 kg) is somewhat lower than what was measured in the tank. If the tank efficiency was based on the mass collected in the tank (536 kg.) instead of the TSS load difference (473 kg.), the TSS removal efficiency would increase by 28 to 34%.

The Multi-Chambered Treatment Train

Fifteen storms occurring from April 29, 1996 through September 8, 1997 were monitored and sampled. Rainfall amounts for these storms ranged from 0.18 to 1.37 inches. Based on the delineated drainage area (0.2 acres), total rainfall volumes ranged from 107 to 815 ft^3. The actual quantity of water passing through the MCTT ranged from 60 to 319 ft^3 and comprised approximately 60% of the rainfall volume. The other 39% of the rainfall volume may have been lost through cracks in the aged pavement surface or in joint leaks between the catch basin and the main chamber. An overestimation of the drainage area may also account for this difference. A consequence of this loss in stormwater was that the unit never surcharged, even though the design hydraulics would have suggested 10 out of the 15 storms monitored should have surcharged and approximately 22% of the total stormwater should have bypassed. This fact made the calculation of overall efficiency problematic.

Total suspended solids influent concentrations from the MCTT ranged from 79 to 1050 mg/l, with a median concentration of 232 mg/l, values comparable to TSS concentrations found in runoff samples at the Stormceptor site. A majority (8 out of 14) of the effluent TSS were below the detection limit. The highest concentration of TSS observed in the effluent was 18 mg/l. The cumulative influent load of TSS to the unit for the 15 consecutive storms was 18.3 kg. The effluent TSS load was only 0.30 kg, making the unit's removal efficiency greater than or equal to 98%. Examination of concentrations at intermediate points, (Pitt et al. 1997) found the majority of the TSS was removed in the settling chamber.

Though the overall TSS removal efficiency was impossible to directly measure due to the water loss problems discussed above, an overall TSS removal was estimated because it is widely used as a key parameter in BMP evaluations. To calculate an overall TSS removal efficiency, it was assumed that any storm volume in excess of the settling tank's capacity (375 ft^3) would bypass the unit. This method resulted in a calculated overall TSS removal efficiency of 78% for the 15 storms monitored.

Total dissolved solids (TDS) influent samples had a median value of 652 mg/l with a range from 164 to 5930 mg/l. The major source of the dissolved solids was a store of road salt located within the drainage area. This salt resulted in a large load of dissolved solids (sodium chloride) to the unit which was 4.5 times the particulate (suspended solids) portion of the solids load for the period of study. The MCTT removed 13% of the TDS load.

Total phosphorus ranged from 0.10 to 0.44 mg/l with a median value 0.25 mg/l. Effluent concentrations were generally an order of magnitude less (median= 0.03 mg/l). This loss signaled a quite high tank removal efficiency of 88%. Dissolved phosphorus was consistently less than 10% of the total phosphorus in both the influent and effluent samples. Dissolved phosphorus removal efficiency (78%), though somewhat less than total phosphorus removal, was still substantial.

Of the five metals examined, total zinc concentrations were consistently the highest (median = 150 mg/l). The removal efficiency for total zinc was 91%. A very high level of removal was observed for all the metals. Because the majority of the total metal concentrations were in the particulate form, the physical removal of this material may be occurring in all three chambers of the unit, although the bulk of the material (associated with the suspended solids) is most likely being removed in the settling chamber.

The removal efficiency of dissolved zinc was 68%, which was somewhat less removal than total zinc. Actual removal may have been greater because effluent concentrations were generally at detection limits.

Total polycyclic aromatic hydrocarbon concentrations (sum of all 16 species) in the influent samples ranged from 2.9 to 23 mg/l. Total fluoranthene and pyrene consistently had concentrations that were more than double the concentrations of other PAH species with median concentrations of 1.8 and 1.4 mg/l, respectively. The total PAH removal efficiency was 94%. The dissolved PAHs averaged 14% of the total concentrations (dissolved PAH conc./total PAH conc.). The only dissolved PAH species which was consistently reported above detection was phenanthrene (median=0.1 mg/l). Because the majority of the PAHs were found in the particulate fraction, most of the removal probably occurred in the settling chamber, which is collaborated by (Pitt et al. 1997). Table 1 is a summary of the tank loads and reduction efficiencies of both BMPs discussed in this report .

Influent and effluent particle-size distributions for MCTT site are reported in Figure 4. Particulate material was comprised mostly of the silt-size fraction (approximately 88%) in both the influent and effluent. Somewhat surprising was the fact that there was no appreciable shift in the particle size distributions between the influent and effluent. Though a decrease in overall particle size would be expected, the size actually increased slightly in the treated water (although the difference was not statistically significant). This fact may suggest that the unit is not selective in the size of particles removed. Another possibility is that material from the filter media, such as sand fines, had escaped around

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Table 1. Tank Mass Loads and Efficiencies of the Studied BMP's

Constituent	MCTT Load-in	MCTT Load-out	% Eff.	Stormceptor® Load-in	Stormceptor® Load-out	% Eff.
TSS	18.3 kg	0.30 kg	98	1420 kg	1040 kg	26
TDS	84.3 kg	73.3 kg	13	37,500 kg	44,700 kg	-19
Total P	19.3 g	2.4 g	88	1.60 kg	1.29 kg	20
Dis. P	0.93 g	0.20 g	78	0.44 kg	0.34 kg	23
Total Zn	11.7 g	1.0 g	91	660 g	520 g	21
Dis. Zn	1.03 g	0.33 g	68	110 g	105 g	5
Total PAH	0.64 g	.039 g	94	67 g	42 g	36

or through the filter fabric. (Pitt et al. 1997) also noted a slight increase in TSS concentrations as the water passed through the filter tank of their pilot-scale unit. Therefore the unit may still be removing larger particles that are subsequently replaced by media material, resulting in no net change in the distributions. It is important to emphasize that even though there was no appreciable shift in particle size distribution, there was still a very high removal of all particulate material (i.e. suspended solids load) in all storms.

Cost-Effectiveness of the Two BMPs

Though the economics of implementing these BMPs was not the focus of this evaluation, it is an important issue to water quality managers, so a brief cost analysis is offered below. Though actual construction costs for the MCTT were $72,000 ($360,000/acre), some of this high cost had to do with contractor's uncertainties in building an unknown device, retrofitting around existing sanitary sewer, and addition of reinforcements for heavy truck traffic. A similar device was built in Minocqua, Wi. for $95,000 ($38,000/acre) which is only a tenth of the cost per acre. Table 2 presents some estimated costs-per-pound of suspended solids removed for the two management practices. Clearly a number of assumptions were required to generate these numbers. Depending on the location and scale, there can be considerable variation in capital and maintenance costs and control efficiencies. For many practices, long-term studies of removal efficiencies have yet to be conducted. Therefore caution must be observed when making these simple comparisons. Even though the MCTT capital and maintenance costs were higher, this was offset by high efficiency of the unit. For the two BMPs studied, the costs per pound of TSS were of the same magnitude.

Summary

This study evaluates the water quality benefits of two retrofit BMPs, the Stormceptor and the Multi-Chambered Treatment Train (MCTT). Both units were placed in public works maintenance yards where automated sampling equipment collected event-mean concentration data. The Stormceptor treated 91% of the total storm volume from 45 storms. Tank reduction efficiencies for the three constituents measured in all 45 storms (TSS, TDS and TP) were 26%, -19%, and 20% respectively. The extremely high salt concentrations found in the runoff water at this site may have compromised the unit's treatment ability. Good agreement was found between calculated TSS load going into the tank (from water quality samples) and what was actually found in the bottom of the tank, although there is some indication that larger-size particles may have been missed in the water quality sampling.

The MCTT was designed for a greater level of treatment and the findings here confirm it. Load reductions of TSS and TP were 98% and 88% respectively. Even dissolved constituents such as dissolved phosphorus and zinc had high removal efficiencies (78% and 68%, respectively). Somewhat surprising was the little change in particle size distribution between influent and effluent, which may of been an artifact of filter media escaping the filter fabric. Similar costs-per-pound of TSS removed were estimated for the two BMPs studied.

Table 2. Cost-per-pound of Suspended Solids to Treat One Acre of Parking Lot for Different Management Options.

Practice	Efficiency	Total Cost	Cost/acre/yr[1]	% Control	Cost per pound removed
Stormceptor®	Tank	$60,000	$479	26	$1.83
Overall		"	"	22	$2.18
MCTT[2]	Tank	$95,000	$1185	98	$1.21
Overall		"	"	78	$1.52

[1] Capital costs amortized over 50 years plus annual maintenance cost.
[2] Based on construction cost at the Minocqua site.

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Assessing the Effectiveness of Orlando's BMP Strategies

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Orlando calls itself "The City Beautiful" for many good reasons. It is the number one tourist destination in the nation. It has affordable housing, high employment, and an abundance of cultural and civic amenities. Its natural beauty has been named in recent polls as one of the greatest reasons for choosing Orlando as a place of residence and business. One of the principal elements of its natural beauty is its lakes. Orlando has 86 lakes, either wholly or partially located within its borders.

Prior to the mid-1970s there were few regulations which protected these lakes. For decades, storm sewers were installed to convey untreated stormwater directly into lakes. Drainage wells were dug for flood protection. These wells linked untreated runoff with the surface underground aquifer, a source of drinking water for many. It was not until the 1970s that it became apparent that stormwater runoff entering the lakes was causing problems to the natural systems.

The lakes, around which some of the most beautiful and oldest homes in the city were built, were showing stresses from decades of stormwater pollution. Symptoms as small as an overabundance of aquatic weeds and as large as massive fish kills indicated that the lakes were reaching their limit of assimilating the influx of contaminants found in stormwater. To address such problems, the city has taken measures to control stormwater pollution and to provide lake enhancements, drainage well protection, and shoreline revegetation.

The Orlando City Code states, in part that, "... the purpose of the Stormwater Utility Bureau and in essence, the stormwater management program of the city is to 1) improve the public's health, safety and welfare by providing for the safe and efficient capture and conveyance of stormwater runoff; 2) authorize the establishment and implementation of a master plan for storm drainage including ... management, operation,... inspection and enforcement; and 3) encourage and facilitate urban water resources management techniques, including ... enhancement of the environment." Orlando's Public Works Department, with its Bureaus of Stormwater Utility, Engineering, Streets, and Drainage, and Project and Construction Management, work to see that these stormwater management goals and issues are successfully addressed.

The Public Works Department's Stormwater Priority Projects List and the Stormwater elements of the city's Growth Management Plan and the Capital Improvement Plan reflect projects which address qualitative (environmental) issues as well as quantitative (flooding) issues.

BMPs are the methods by which qualitative issues are being addressed. A BMP, or a best management practice, is generally defined as the most effective, practical means of preventing or reducing the amount of pollution generated by non-point sources, such as stormwater runoff, to a level compatible with water quality goals. A BMP should take into account problem assessment, outcome alternatives, public input, and technological, economic, and institutional considerations. The city incorporates BMPs routinely. Some are very basic, while others are more complex. Examples of BMPs utilized by the city include:

Street Sweeping - Street sweeping can be a useful BMP incorporated by a city. Street sweeping removes pollutants, sediments, leaves, and debris from the street surface before they can be flushed into a receiving water body. Orlando sweeps over 40,000 curb miles per year. There are 72 residential routes, seven industrial/commercial routes, and five downtown/near downtown routes. Over 27,000 cubic yards of material is removed annually by the street sweeping program. This material is taken to a landfill for disposal instead of being washed off of the streets in stormwater and deposited into the city's lakes.

Retention/Detention Ponds - In 1984, the City of Orlando implemented rules contained in the *Orlando Urban Stormwater Management Manual* and became one of the first Florida municipalities to require both retention and detention facilities for new development. Under these rules, developers must physically separate the retention volume from the detention volume in what are referred to as "two pond" systems. All development must include pollution abatement capabilities to treat either 1/2 in. of runoff or

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runoff generated by a one-inch rainfall, whichever is greatest. Treatment is defined as 1) retention with no discharge and/or 2) detention with an approved filter discharge. A detention facility for a developed site must be capable of controlling the runoff volume expected from a 25-year frequency/six-hour duration storm. Detention volume may be discharged, but only at a rate not to exceed the peak rate of discharge of the site prior to development.

Lake Revegetation - Twenty-two miles of shoreline in 45 of the city's lakes have been planted with native aquatic plants. Revegetation returns the lakes to a more natural state and better enables lakes to assimilate nutrient inputs. This also provides valuable wildlife habitat. Lakefront property owners are also encouraged to plant native aquatic plants. City staff believe that revegetation enhances lake aesthetics and water quality. Plantings are required in wet detention facilities. These ponds are shaped with a littoral shelf to facilitate plantings.

Pollution Control Devices (PCDs) - Pollution Control Devices are screening structures that are placed at stormwater outfall pipes to prevent trash from entering the lake. The City of Orlando has installed over 250 PCDs. Cleaning debris from the PCDs after large storms is important and very labor intensive. If the PCDs are not cleaned out, the buildup of leaves and debris can cause the device to be "blown out" by heavy rainfall events, thereby requiring maintenance and repair.

The most unique PCD the city has is a trash collection device recently installed at one of its largest outfalls. At Lake Rowena, a 108" outfall delivers the runoff from a 844 acre drainage basin. Debris that is flushed through this system is so extensive that the normal screening processes became ineffective in medium storm events. Therefore, the city borrowed from the wastewater field and installed flow-actuated moving bar screen (constructed 20 feet below the ground) to intercept debris prior to discharge into Lake Rowena. As flow, and thereby debris, begins to move through the system, the moving screen begins to operate. The screen catches debris, lifts it up, and deposits it into a hopper. The hopper is periodically vacuumed using vactor truck.

Removal of the trash and debris is only the first line of defense in the struggle for pollution abatement. To combat the other deleterious effects of stormwater, runoff has to be treated. The City of Orlando has incorporated many innovative BMPs to treat stromwater, most with great success.

Notable examples of innovative BMPs used by the city include alum treatment systems installed at four of the city's lakes, a Vertical Volume Recovery System, a Packed Bed Filter Project, an Urban Wetland Systems at Lake Greenwood and the La Costa canal, and the Lake Wade Periphyton Filter Water Garden.

Lake Dot Alum Treatment System - Lake Dot is a small (6 acre) lake in downtown Orlando, with a drainage basin of almost 300 acres. The lake had received untreated runoff from this highly developed watershed for decades. Largely as a result of the influx of pollutants during rain events, the lake had experienced periodic fish kills due to low dissolved oxygen. In 1988, construction of the Orlando Arena (home of the NBA's Orlando Magic) within the Lake Dot drainage basin was being planned. When considering measures to meet the city's stormwater treatment requirements, planners first considered surface retention/detention ponds. Because the cost of acquiring land needed for constructing the ponds was prohibitive, an alternative approach was selected in which alum (aluminum sulfate) is added to stormwater to tie up nutrients and suspended solids and precipitate them to the bottom of the lake. Since 95% of all runoff input enters Lake Dot through a single storm sewer line, the same line that the Arena retention ponds would have used as a discharge point, it was determined that the entire amount of stormwater conveyed through this line could be treated with alum. This system has been in operation since 1989, and with the exception of a few operation difficulties, has operated satisfactorily. Improvements to water quality have been significant. Prior to installation of the Alum Treatment System, the lake had been eutrophic, with a Trophic State Index (TSI) averaging over 60. Since the installation of the system, Lake Dot has been classified as mesotrophic, with a TSI average of just over 50.

The success at Lake Dot has encouraged the use of alum treatment at other locations. Alum treatment systems have been installed at Lake Lucerne in downtown Orlando, Lake Holden (as part of a joint project with Orange County), and Lake Rowena. An alum treatment system is currently being planned for Clear Lake.

The Vertical Volume Recovery System - The Vertical Volume Recovery System (VVRS) combines in-pipe storage, a sump device for sediment removal and a sand filter for fine sediment and dissolved pollutant removal. The stormwater treatment system was developed as a part of an inter-basin diversion plan needed to relieve flooding from Lake Olive, one of our downtown lakes. The system was designed to treat one inch of runoff from the roadway surfaces within the basin area. It was believed that the sand filter would be an effective method of stormwater treatment where land is too expensive for standard retention/detention. To date, the VVR Ssystem has been a maintenance burden. The treatment capabilities of the sand filter have been disappointing.

Monitoring results showed some removal of dissolved and particulate pollutants within the sump device. The sump removed lead at rates between 50 and 78%. Zinc was removed at rates between 40 and 56%. Cadmium was removed at 50 to 60%. No significant removal of other heavy metals was observed. Total Kjeldahl Nitrogen (TKN) was removed at rates ranging from 14 to 56%. Total phosphorus was removed at rates from 29 to 51%. Total suspended solids were removed at rates between 14 to 71%.

Monitoring results also showed very poor removal of dissolved and suspended pollutant types by the filter. Removal

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was 3% for copper, 5% for nitrate-nitrite, 10% for total phosphorus and 12% for suspended solids. Export of pollutants was observed at 10% for lead, 248% for ammonia, 2% for TKN and 19% for orthophosphate.

The Packed Bed Filter Project - A Packed Bed Filter Project at Clear Lake has proven to be very successful. The idea of using vegetated rock media filters to treat stormwater was proposed in response to growing concern over the water quality of Clear Lake. Clear Lake, once a lake that was clear, had become eutrophic by the early 1980's, and was one of Orlando's poorest quality lakes.

The Clear Lake drainage basin consists of over three square miles of highly developed urban area. Stormwater runoff from this basin is conveyed into Lake Beardall and Clear Lake. The Packed Bed Filter treats the initial storm runoff from about 160 acres. Because the project demonstrated an innovative technology, the city was successful in obtaining a grant from Florida Department of Environmental Protection (FDEP). The project uses a proven wastewater technology to urban stormwater pollution. Simply put, the system consists of a packed bed filter, similar to a trickling filter, and hydroponic aquatic plants.

Ten beds were established, including five crushed concrete and five using granite. Four different types of native aquatic plants were planted in four of the concrete and four of the granite beds. Two beds were set aside as controls. Assessments were performed on the effectiveness of the overall packed bed filter system, the performance of the individual beds, and the best rate at which to operate the system.

Monitoring results indicated medium to high effectiveness in treating stormwater. Total phosphorous, total suspended solids, and volatile suspended solids and cadmium were removed by the overall packed bed system at a rate exceeding 80%. Total nitrogen, TKN, nitrates, nitrites, chromium, copper, lead, and zinc were removed at rates ranging from 25 to 80%. Removals of ammonia, orthophosphate and total dissolved solids were not as effective, with only 6, 14, and 8% removal, respectively.

The Greenwood Urban Wetlands - What was initially conceived as a flood protection project, ended as a multipurpose, multi-use facility. Florida's first stormwater treatment train was created to provide stormwater quality improvement, address the need for additional park area, and provide flood protection for a 522-acre drainage subbasin. The project required the expansion of Lake Greenwood to over six times its original size. It also involved planting over thirteen acres of wetland and upland vegetation, and constructing footpaths, bridges, and stormwater control facilities. In addition to these features, the new man-made wetland includes a pumped irrigation system which utilizes stormwater runoff, instead of potable water to irrigate the park and a neighboring city-owned cemetery. FDEP provided a grant to determine the effectiveness of the created wetland stormwater treatment. The results were very good, and would likely have been even better had it not been for a high rate of groundwater inflows. This award winning project os one of Orlando's most significant stormwater management success stories.

This Greenwood project has proven to be a success in all areas it had been intended. It provides flood protection, stormwater treatment, a much-needed passive park, and stormwater reuse through irrigation. The city has repeated this concept, on a slightly smaller scale, with the construction of another urban wetlands, the La Costa Urban Wetlands.

Periphyton Filter Water Garden at Lake Wade - This joint venture project with the St. John's River Water Management District will demonstrate the effectiveness of a managed growth periphyton filtration system. The Periphyton Water Garden is a stair-stepped concrete structure with a surface that fosters the growth of algae. Water from Lake Wade is pumped to the top of the structure and flows over the algae and back to the lake. The algae consume nutrients and traps sediment. Each week, the algae are harvested from the water garden and trucked away from the site. The algae biomass can be used as an environmentally friendly packaging material or be spread on park grounds or ballfields as a fertilizer.

Periphyton, or attached algae, comprise the most productive component of a wetland system. The Periphyton Filter accelerates natural algae growth processes in a controlled farmed system. A periphyton filter delivers a thin sheet of untreated water over a diverse community (100-200 species) of attached algae. Since techniques and devices developed for harvesting the algae leave "roots" intact, rapid regrowth occurs. Algal species diversity and frequent harvesting make periphyton filters adaptable to changing conditions, for consistently high productivity.

Periphyton Filters work on a wide range and various levels of contaminants. Particularly interesting is their ability to function at extremely low levels of pollutants, which lends itself to lake restoration and treatment. The periphyton filter is a promising approach to reducing non-point source pollution in Lake Wade.

This project went on-line in December, 1997. The Water Management District and the City will be conducting research and monitoring during 1998 to determine its effectiveness.

Orlando is a leader in developing innovative approaches to stormwater treatments for quality and quantity control. The City of Orlando is very proud of its stormwater management practices.

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Evaluating Public Information Programs: Experiences with the Florida Yards and Neighborhoods Program

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This report provides an overview of the evaluation methods applied to the Environmental Landscape Management (ELM) and Florida Yards and Neighborhoods (FYN) programs of the University of Florida's Cooperative Extension Service. Florida Yards and Neighborhoods is a public education component of the major state-wide Environmental Landscape Management program.

The Homeowner Research Questionnaire has been a useful tool in measuring the level of adoption of environmental landscape management practices by different program groups. County-based surveys are used to show overall program effectiveness and to compare different program delivery methods. A 1997 analysis by the University of Florida comparing three delivery methods used throughout Florida, indicated that educational seminars are an essential component in helping people adopt ELM practices.

Social marketing research conducted in 1997 studied the perceptions that Tampa Bay homeowners have regarding environmental landscape management practices. This research included pre- and post-test evaluations of educational materials used by Florida Yards and Neighborhoods.

In this report, a brief overview of the ELM and FYN programs will provide the foundation for discussing the evaluation. This overview is followed by a description of two ELM/FYN evaluation methods. This paper summarizes, in part, two reports prepared in 1997: *Adoption of Landscape Management Practices by Florida Citizens,*[1] by Glenn D. Israel, Janice O. Easton and Gary W. Knox; and *Social Marketing Study of the Perceptions Tampa Bay Homeowners have Regarding Environmental Landscape and Protection Practices,*[2] by Bonnie Salazar. Both are cited at the end of this paper. The concluding section will look at lessons learned and future plans for measuring program effectiveness.

Introduction

Evaluating community-wide environmental education programs is essential to measuring the effectiveness of outreach efforts. Determining program effectiveness not only aids in refining educational tools, it is often essential for obtaining or maintaining funding. However, measuring changes in awareness, knowledge and behavior presents a variety of challenges.

Those implementing community-based educational programs do not have the advantage of a captive audience who must come to class and turn in assignments. In fact, getting the target audience to respond to measurement techniques is one of the biggest challenges of all, even if it is a list of questions about their yard design and care practices.

Overview of ELM and FYN

Environmental Landscape Management was developed in the late 1980s to educate professionals and homeowners on how to create attractive, healthy landscapes by taking an ecosystem approach to landscape design and maintenance. "Right plant, right spot" is the key tenet. Educational materials emphasize adopting proper cultural practices to reduce landscape problems and negative impacts on the environment.

Florida Yards and Neighborhoods is a component of the ELM major state program. Developed in partnership with the Sarasota and Tampa Bay Estuary Programs, FYN increased the emphasis on the problems of stormwater runoff pollution and the need to protect estuary systems. Since its implementation in 1992, Florida Yards and Neighborhoods has expanded its scope to include freshwater systems.

A variety of educational tools have been developed for the Florida Yards and Neighborhoods Program, many of them initially funded by the Sarasota Bay and Tampa Bay National Estuary Programs. As the program continues to develop, the need for additional tools becomes apparent, as does the modification of some of the existing ones. A continuing goal for those involved in the FYN Program is obtaining the funds to purchase existing materials and develop new ones.

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A Guide to Environmentally-friendly Landscaping: The Florida Yards and Neighborhoods Handbook[3] (Florida Cooperative Extension Service, 1994) is in its third printing. *The Florida YardStick Workbook*[4] (Florida Cooperative Extension Service, 1998)[4] supports the handbook by providing a checklist of environmentally friendly landscape design and care actions that are discussed in the book. Two slide presentations demonstrate ELM/FYN practices and discuss the benefits of adopting them: "Creating Your Florida Yard" and "Maintaining Your Florida Yard." The FYN message is presented in video form through the half-hour *Reclaiming Paradise: Florida Yards and Neighborhoods.*

Various national estuary programs in Florida have provided funding to implement the FYN program in their counties. The purpose of this short-term funding was to help establish a successful program that local government entities would use to help meet their goals of water conservation and reduced stormwater runoff and solid waste. Program evaluation plays an important role the continuation of an FYN Program in a county.

Program Evaluation Approaches

Program evaluation has always been an important component of the Environmental Landscape Management Program. A self-report survey using the Homeowner Research Questionnaire is the primary state-wide evaluation tool. In 1997, a social marketing research project provided additional methods, on a regional basis, for measuring and increasing program effectiveness.

The Homeowner Research Questionnaire Survey

The Homeowner Research Questionnaire measures the number of environmental landscape management practices the respondent is using. Participants complete the pre-test questionnaire prior to receiving an educational program or educational resources. Ideally, the post-test questionnaire is completed six months later. Comparisons of pre- and post-test responses measure changes in behavior.

The pre-test questionnaire contains the following categories: General Information, Site Analysis, Stormwater Runoff, Irrigation, Fertilizer, Pest Management, Wildlife, Information Sources and Waterfront. With the post-test questionnaire, the Information Source category is dropped and questions are added under the new category "Views on FYN Practices." Landscape practices include using slow-release fertilizers, applying one pound of nitrogen or less per 1,000 square feet, leaving grass clippings on the lawn, tolerating some pest damage, watering according to season and directing downspouts into the lawn or plant beds.

Each questionnaire is coded according to the type of program the participant received. Program types can include the following: neighborhood program, all-day workshop, one-hour presentation, exhibit, and Master Gardener training. In some counties, people also receive publications by coming into the Extension Service or by calling and requesting information. This category is referred to as "Publications only."

Each county enters its own data using a standard template. The data is sent by computer disk to be analyzed by the Program Development and Evaluation Center of the Institute of Food and Agriculture Services, University of Florida. Counties use the survey data to evaluate different delivery methods and in their annual reports to funding sources. Because the program content and landscaping practices taught among the counties are consistent, statewide comparisons of delivery systems can also be made.

In 1997, the University of Florida Cooperative Extension Service conducted a state-wide comparisons of three ELM/FYN program delivery methods: Master Gardener training, 1-6 hour seminars and publications only.[1] A control group was composed of non-participant residents who completed the ELM Homeowner Research Questionnaire in 1993 to establish a baseline. The data for the program delivery groups was collected immediately prior to and approximately six months following educational programs conducted during the 1997 Fiscal Year (July 1, 1996 through June 30, 1997.)

This study looked at average adoption rates in the different groups, as well as the adoption rate for specific landscape design and care practices. However, this paper will discuss only the findings regarding overall adoption rates.

The Master Gardener group data was collected from 134 residents in nine counties. Three hundred and twenty residents in six counties contributed the data for the 1-6 seminars group. In two counties, data were collected from 72 residents who received publications only. The Master Gardeners had the highest response rate, with 63% returning their surveys, compared to a 42% response rate for seminar participants and 57% for the non-equivalent comparison group. This study focused on 39 landscape care and design practices included in the questionnaire.

A comparison of the pre- and post-test survey results indicated an increase in the average number of practices used by participants of all three program delivery methods, while non-participants remained essentially unchanged (Figure 1).

The results of an analysis of variance show that while the type of property, person maintaining the property, presence of a permanent irrigation system and parcel size did not affect the adoption of practices, the type of program did ($F=31$; $p=0.001$). A multiple comparison of adjusted means for net adoption using Fisher's least significant difference indicated that each program type differed significantly from the other groups in the number of practices adopted (Table 1).

Social Marketing Research

Three grants funded social marketing research on the Florida Yards and Neighborhoods/Environmental Landscape Management Programs in 1997. Funding came from

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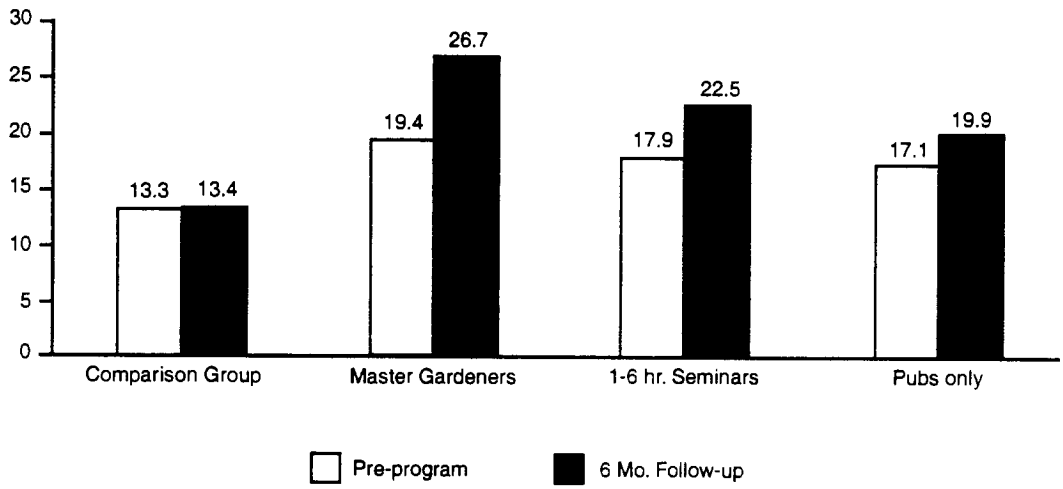

Figure 1. Mean numbers of Best Management Practices used by program type.

Table 1. Average Number of Environmental Landscape Management PracticesAdopted by Type of Program

Type of Program	Mean Number of Practices Used Before the Program	Net Number of Practices Adopted	Adjusted Net Number of Practices Adopted[1]
Master Gardener	19.4	7.3	6.9
1-to-6 hr seminar	17.9	4.5	4.3
Publications only	17.1	2.8	2.6
Comparison group	13.3	0.1	0.0

[1] Model of R^2 of .135 and p=.001. Adjusted means were generated by the analysis of variance for comparison using Fisher's least significance differences.

the following: Florida Cooperative Extension Service; NPDES (National Pollution Discharge Elimination System) Education Subcommittee of the Florida Department of Transportation, District 7; and West Coast Regional Water Supply Authority (WCRWSA). In-kind services, including co-moderating focus groups, were also supplied by the Cooperative Extension Service. The research was conducted by Chastain/Skillman, Inc.

Social marketing adapts commercial marketing technologies to programs designed to influence voluntary behavior that improves the personal welfare of target audiences and their society. The purpose of this research project was to determine the perceptions Tampa Bay homeowners have regarding environmental landscape and protection practices. There was the sense in entering this social marketing research project that there are different consumer groups relative to yard care. It was hoped that the research would help identify those groups by the benefits and drawbacks they perceived in the ELM program. FYN could then target those groups that would be the most receptive by crafting programs to meet their needs.

The research included focus groups, participant-observation, surveys and the pre- and post-testing of educational materials. The Florida Cooperative Extension Service, especially the Hillsborough County office, coordinated the effort and provided in-kind services. This paper will look at the pre- and post-testing of program materials.

The *Florida YardStick Poster* was introduced into the Florida Yards and Neighborhoods Program in 1994. This colorful poster groups important yard care and design practices (or actions) into seven categories. Each Florida Yard action is worth a given number of inches (or credits) and once a homeowner has accrued 36 inches or more, his or her yard can be certified as a Florida Yard. The poster is 38 inches tall by 24 inches wide.

The poster was useful in classroom and workshop settings. It graphically showed the relationship between taking action and creating an environmentally friendly Florida Yard. However, there was some question as to its usefulness as a tool for homeowners. There was a suggestion to turn the poster into a workbook format—keeping the actions, but changing the format and adding practical "how-to" information to augment the handbook's general principles.

It was decided to conduct a post-test evaluation of the workbook during the initial focus groups conducted as part of the social marketing research. Toward the end of the

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focus group meeting, participants were asked to look at the poster and respond to questions about its appearance, usefulness, intended audience, etc. After the general discussion, participants were given sections of the poster to evaluate. They wrote their comments and questions about the section they received.

In general, participants said the design was friendly and cheerful. It was noted that for a poster, the print should be larger. Often, when asked who the intended audience was, participants said they thought it was for school children. Some said they would hang the poster in their garage as a reference. A frequent suggestion regarding format was to change it into a booklet format. Recommendations regarding text included minimizing the use of technical terms. Participants indicated that words such as "low-maintenance," "irrigation," "least toxic," and "pervious" were hard to understand.

The Florida YardStick Workbook incorporated the colors and graphics used in the poster because of the positive reaction people had to these elements. The book was sized to fit into the back flap of the FYN Handbook. The basic concept of the poster, with the categories and actions was maintained. A major change was in the use of terminology. When possible, technical terms were replaced with more common terms. For example, "water sprinkler" replaced "irrigation system."

The pre-test evaluation of *The Florida YardStick Workbook* was conducted using the first draft of the workbook. Because of time restrictions, the pre-test was not done in focus groups. Instead, through one-on-one interviews, people seeking to enter Master Gardener training were asked about their perceptions regarding the workbook.

Again, the general response was that the overall appearance is attractive and friendly. Participants found the information useful and the writing easy to understand. Many recommended that a larger print size be used. A common complaint was that the pages looked cluttered.

In response to these observation during the pre-test evaluation, a larger print size was used and more white space was added. Because of cost constraints, information was condensed or deleted. However, this process of honing the content helped define critical elements.

Conclusions

Public information programs are viewed as an essential component in helping to reduce stormwater runoff pollution, conserve water resources, protect wildlife habitats, and engage in other environmental protection actions. Evaluating environmental information programs is a challenging but essential process for determining the effectiveness of various strategies, for developing new approaches and for justifying new, additional or continued funding.

In 1997, the University of Florida Cooperative Extension Service used two evaluation methods with its Environmental Landscape Management Major State-Wide Program, which also encompasses Florida Yards and Neighborhoods. The ELM/FYN Homeowner Research Survey proved to be a useful tool for comparing the effectiveness of three different program delivery methods. Taking a social marketing approach to evaluating educational publications was a productive method for making them easier to read and comprehend.

Of course, it is important to evaluate the evaluation mechanisms. The ELM team is currently in the process of refining the ELM/FYN Homeowner Research Questionnaire. Currently, the questionnaire measures changes in self-reported behavior. An important question is to what extent does that behavior change result in water conserved, reduced nutrient runoff from yards, a decrease in the use of toxic pesticides, or other quantifiable results? Therefore, one goal in revising the questionnaire is developing queries that can result in valid, quantifiable answers.

Another reason to have an ongoing process of evaluating the evaluation techniques is that new research results regarding stormwater runoff pollution or other environmental issues may lead to new goals for public action. For example, early educational efforts in public information programs regarding stormwater runoff pollution focused on having homeowners reduce their use of fertilizers and pesticides and keep their driveways and street gutters clean. However, with research showing the increasing importance of atmospheric nitrogen deposition, we may want to focus on additional behaviors, such as increasing public support for stormwater retrofit projects, street cleaning, and mass transportation.

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Examining the Need for Project Evaluation

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Abstract

Retrofit project evaluation is as critical as project goals and objectives. Formulation of a project evaluation should begin when the project begins; it is an essential part of the planning process. Without an evaluation system in place, it is likely that you will waste precious time and funding. Many assume that evaluations are expensive and require extensive expertise. Sometimes this is true, but there are easier and less expensive approaches that can be used, if you accept the tradeoffs that come with a less complicated evaluation method. Evaluations make it easier to make a decision and justify your choices. This article presents the role of evaluation, describes several types of evaluations and provides examples.

Introduction

Too often project evaluations are scheduled at the end of a project, and incompletely determine whether the goals and objectives have been met. Evaluations should instead be an ongoing part of any retrofit project or program (henceforth project means both project and program). The typical project, designed to accomplish a specific task, is commonly evaluated by the completion of just that task. In addition to accounting for whether specific activities are being implemented, and resources expended, evaluations need to be designed so as to determine the impacts over the whole project. One should also evaluate the broadest impacts caused by completion of the task. For example, a recent outreach program to developers and homebuilders focused on ways to achieve development and still protect wetlands. The program was evaluated only by the number of presentations, the amount and type of material distributed, and the requests for information. The program was not evaluated against wetland loss prevented, so the program's entire impact was underestimated.

Most completed nonpoint source project evaluations are inadequate. Many projects do not even include an evaluation. In a study on river conservation enhancement, Holmes (1991) found that only five of almost 100 projects had post-restoration evaluations. The most common excuses given for not carrying out evaluations were time, cost, "evaluations not worth it" and "evaluations are difficult."

Most completed evaluations also fail to determine whether the project had the long-term impacts it was designed for. For example, most monitoring associated with nonpoint source control projects focuses on the number of Best Management Practices (BMPs) implemented, and stops when implementation funds have been expended. The Rural Clean Water Program (RCWP) showed the importance of lag time between when BMPs are implemented, and when water quality changes (USEPA, 1993). Linking project evaluation to BMP implementation funding creates an inherent bias toward failure to demonstrate water quality improvements, because it may take years between BMP implementation and the realization of water quality improvements.

In response to this lag problem, and the need to have a national evaluation on the effectiveness of the Section 319 (Nonpoint Source) Program (EPA, 1991), the United States Environmental Protection Agency (USEPA) created the Section 319 National Monitoring Program. That program established frameworks to more carefully document water quality impacts associated with selected BMP implementation, especially over a longer period of time than usual. The National Monitoring Program's objectives are1) to scientifically evaluate the effectiveness of watershed technologies designed to control nonpoint source pollution, and 2) to improve our understanding of nonpoint source pollution (USEPA, 1991). For more information on the National Nonpoint Source Monitoring Program see Osmond, et al. (1997).

Why Evaluate?

While evaluation is both an art and science, it is critical to the long-term success of any project. Project evaluations allow managers to build on success and learn from mistakes. Beech and Drake (1992) say it best; "Would you invest your own money knowing that you would not get any feedback on performance of the investment?" Without an evaluation system for your project, you could be wasting time and money and not know it. The Highland Silver

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Lake RCWP Project is an excellent example of why evaluations are essential to the success of a project. Designed to address lake sedimentation, the project's monitoring and evaluation efforts showed that the lake was instead being impacted by natric soils (causing high turbidity levels) and not excessive sediment loading (Davenport & Kelly, 1986). Evaluation systems can also play an important role in linking your activities to the public and gaining their support.

Evaluation Types

Evaluations can be done at different stages during a project, or at multiple stages. There are many evaluation methods, techniques, and approaches designed for specific purposes and project phases (planning, implementation, and evaluation). A comprehensive evaluation will utilize a combination of these methods to accurately evaluate a project. The four major types of evaluations are Formative, Process, Outcome, and Impact. After discussing these in the next paragraphs, we will continue with discussion of more modest forms of evaluation.

Formative Evaluation (before the project starts) - A formative evaluation is undertaken to test approaches, materials, and ideas. Additionally, formative evaluations are used to understand the target audience before a project is implemented. For projects involving communication, a key factor is determining the appropriate target audience, based on project goals and objectives. A second sort of formative evaluation is a needs assessment. The principal difference between the traditional formative evaluation and needs assessment involves the decisions that arise from the outcomes. According to Herman and others (1987), needs assessments result in better allocation of resources to meet high priority needs. The formative evaluation is an attempt to improve implementation prior to getting started. For example, school materials about wetlands might be selected by an advisory group of experienced teachers. Or, land owners could be surveyed concerning barriers to adopting conservation practices.

Process Evaluation (during)- While some consider this part of a formative evaluation, for this paper it is separate and focuses on tracking activities. Bean-counting is another phrase for this type of evaluation. Using process evaluation, managers can monitor implementation activities and provide timely information to their staff and project sponsors. Process evaluations allow managers to change project activities in response to ongoing feedback. Major problems in using process evaluations for nonpoint source projects include the inability of process evaluations to determine the cause of the problem (before implementation begins), and the lag time between implementation and reporting of accomplishments. Process evaluations can 1) monitor the program activities by recording activities systemically, 2) keep everyone focused on the big picture, and 3) provide a database to allow project managers to evaluate cost-effectiveness at every stage of the project. Examples of process evaluation are tracking BMP implementations and requests for technical assistance.

Outcome Evaluation (afterwards) - This is also referred to as a "summative" evaluation and measures the short-term results associated with a project. The big question, "What did I get for my money?" gets answered here. Outcome evaluations can be used to 1) measure changes in knowledge, attitudes, awareness, skills, aspirations (KASAs) or behavior, 2) determine whether the project worked within the desired time frame, and 3) determine whether the project goes beyond the desired effects. An example would be a workshop evaluation that focuses on participant changes, or the USEPA Region 1 review of their Section 314 Clean Lakes Projects (Metcalfe & Eddy, 1992).

Impact Evaluation (much later)- Impact evaluation is the most difficult type of evaluation to complete. It measures long-term impacts. This type of evaluation requires durability of project goals and objectives. It is important to note that the world keeps changing as your project progresses, so evaluators must determine whether the project's goals and objectives have remained constant. Longer-term results usually vary from short-term results, so impact evaluation is needed to measure the ultimate value of the project. This type of evaluation is extremely important for pilot and demonstration projects. Many programs are developed and implemented based on either "process" or "outcome" evaluations of pilot or demonstration projects and lack real impact evaluation.

EPA's Clean Lakes Program (Clean Water Act Section 314) stresses evaluation through all three phases: Diagnostic, Implementation, and Post Restoration. Phase 1 evaluations should be the least complex (formative and process), while Phase 3s are more complex (impact evaluations that provide individual outcomes and future direction). One Phase 3 Post-Restoration Monitoring study was conducted five years after the successful completion of a Phase 2 lake/watershed management program designed to eliminate watershed sources of pollutants to Lake Le-Aqua-Na and improve in-lake water quality. Phase 2 of the project was judged a success after evaluation indicated the project met its immediate goals and objectives. The Phase 3 study, five years later, confirmed that the BMPs were still in place, that the effectiveness of some of the BMPs had decreased but were still achieving their goal, and that the overall management program was still having the desired effect on the lake (Davenport & Kaynor, 1998).

Clean Lakes Projects and Nonpoint Source Control Projects are similar in that they rely on changes in human behavior, but the basis of determining which changes are needed is different for each. Proposed changes in Clean Lakes Projects are based on stakeholder-involved diagnostic/feasibility studies. Nonpoint Source Project changes are usually prescribed based on preconceived or documented problems without stakeholder input during the formulation phase. This is an important difference which needs to be considered in all outcomes or impact evaluations.

In Clean Lakes Projects, landowner/operator education efforts start during Phase 1, their reactions and KASA levels are designed into the Phase 2 implementation plan. With Nonpoint Source Projects, landowner/operator reactions and KASA levels are used to modify the implementa-

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tion plan after it is underway. When a Clean Lakes or Nonpoint Source Project is promoting BMPs, evaluating education being used is especially critical to accomplishing long-term behavior changes. Too often NPS projects judge success only by the number and types of BMPs that landowners/operators are willing to install. Long-term maintenance and operation are seldom considered within Nonpoint Source Project evaluations, but are a major focus of Phase 3 Clean Lakes Projects.

Evaluation Components

There are many different evaluation components available for Nonpoint Source Project evaluation (Table 1).

A brief summary (modified from Beech & Drake, 1992) of each of the Hierarchy's components follows:

1) Inputs - Project resources that are used to carry out the work. These resources include, at a minimum, funds, paid staff, volunteers, office space, and supplies.

2) Activities - Actions that are done, to implement the work, such as planning.

3) Target Audience - Identifying who the target audience is for the project, and how they influenced project design and implementation.

4) Reactions - Reactions of the target audience to the project activities.

5) KASA Change - The project's activities must change either the knowledge, attitudes, skills, or aspirations of of the target audiences (participants) to get implementation.

6) Changes in Behavior - Participant behavior changes through the implementation process.

7) End Results- Results compared to the goals and objectives developed when the activities were planned.

A modification of Bennett's Hierarchy of Evidence for Program Evaluation is very relevant to Nonpoint Source Projects (Table 2).

Evidence of real impact becomes stronger as the evaluation ascends the hierarchy. The two lowest levels (inputs and activities) provide little or no measure of participant benefit or environmental improvement. Level 3 provides the first opportunity to get an indication of educational possibilities. If a focus of the evaluation is to increase project performance, it is important to apply more evaluation techniques at the lower levels (3 and under). Thorough assessment of project effectiveness requires evaluations to be conducted at the upper levels of the hierarchy. Evaluations covering Levels 4 (Reactions) and 5 (KASA) provide an indication of whether or not the activities are working. KASA changes give an indication of potential BMP adoption. External accountability increases as the level of evaluation increases, while internal accountability follows the reverse trend. Since the project is likely to change as it develops, project evaluation techniques selected for higher levels of the hierarchy must be flexible enough to respond to change.

Table 1. Bennett's Hierarchy of Evidence for Program Evaluation (Beech & Drake, 1992)

Level	Component
7 (highest)	End Results
6	Behavior Change
5	KASA Change
4	Reactions
3	Target Audience
2	Activities
1 (lowest)	Inputs

Table 2. Modified Bennett's Hierarchy

Level	Component
7 (highest)	Behavior and Resulting Environmental Change
6	End Results (linked to funding period)
5	KASA Change
4	Reactions
3	Target Audience
2	Activities
1 (lowest)	Inputs

The level at which an evaluation is carried out (i.e., 1 to 7) has a tremendous impact on cost, requirements, and useability. The higher up the hierarchy, the greater the probability that project results will be influenced by external factors which are more difficult to evaluate. The evaluations that utilize the higher levels of the hierarchy usually involve more expensive data collection and more time to obtain results. Also, more expertise is required to design evaluations, analyze data, and provide feedback. In addition, evaluation techniques require expertise to separate external influences from actual project.

Reversing the order of Levels 6 and 7 is more appropriate for Nonpoint Source Projects. The resulting evaluation then focuses on surrogates for water quality, and completing the implementation plan. This modification allows evaluators to determine if the BMPs are being maintained and operated properly after the agencies have moved on to other priority areas, and whether water quality has improved. It also takes into account the "Hawthorne effect" where participants respond favorably about implementation because of attention rather than commitment to change (when attention has faded they revert to previous behavior). The modification also attempts to link long-term behavior change with long-term changes in the environment that are associated with the behavior in question. The modified Hierarchy, shifts the focus of the evaluation to long-term behavior changes after the implementation of BMPs has been completed.

Table 3 relates the level of evaluation to the type of evaluation. Based on cost and complexity, project managers

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Table 3. Modified Hierarchy Levels as They Relate to Evaluation Type

Level	Component	Formative	Process	Outcome	Impact
7	Behavior Changes		X	X	X
6	End Results		X	X	X
5	KASA Change		X	X	X
4	Reactions		X	X	X
3	Target Audience	X	X		
2	Activities	X	X		
1	Inputs	X	X		

must decide what type of evaluation is appropriate to document their overall project or phase. While not all projects need the same level of evaluation, the evaluation should be planned prior to initiation, and not as an afterthought. The level and type of evaluation utilized must be linked to program needs. For example, if the program status is needed, an evaluation that focuses on Levels 1 and 2 of the hierarchy would be sufficient. If, however, program managers are contemplating changing the direction of the program or enhancing specific components, the evaluation must be able to correlate data from Levels 1 and 2 evaluations to target audience (Level 3), reactions (Level 4), KASA (Level 5) and end results (Level 6) for the existing efforts, and what would be expected with future efforts.

Table 4 shows what level of the modified Hierarchy should be addressed during each project phase. Using Table 4, managers can design project evaluations that build from one phase of the project to the next. Again, the Clean Lakes Program is an excellent example of this process.

Based on earlier experiences in agricultural nonpoint source program evaluations (Model Implementation Program, Agricultural Conservation Program Special Water Quality Projects) and the Clean Lakes Program experience, the third generation of USDA's nonpoint source program (RCWP) built evaluation components into the process. The premise of RCWP was that agricultural nonpoint source pollution could be controlled at the farm scale to cost-effectively improve off-site water quality. The Comprehensive Monitoring and Evaluation (CM&E) component of the RCWP was established to document the farm level effectiveness of the program, based on a sample of 20% of the projects funded under RCWP. CM&E Projects were selected for intensive monitoring and evaluation efforts to represent the universe of agricultural nonpoint source problems being addressed. The evaluation framework for RCWP consists of three tracks: 1) individual project monitoring, 2) CM&E projects and 3) national surveys. The individual projects were primarily evaluated in terms of inputs and activities (Levels 1 & 2) outlined in the implementation plans. Attempts were made to link these process evaluations with project outcomes. These evaluations (tracking activities) were carried out by the project technical and administrative staff as part of their ongoing duties. The process evaluations were inexpensive and very straightforward. In some of the individual projects, process evaluations were supplemented by Cooperative Extension Service surveys of the target audience (Level 3) to estimate changes in knowledge (Level 5) and document reactions (Level 4) to the program. CM&E components separately fund (usually more than $1,000,000 per project) staff and work plans; they were integrated as part of the individual projects so the project's technical and administrative staffs were involved in the evaluation but not responsible for it. CM&Es integrated all four types of evaluations in the individual projects over a 10-year period. Formative evaluations were used with the watershed land owners/operators to help determine resource allocations between project components and to establish a baseline (Levels 3 and 4) to document behavior patterns (Level 6) and knowledge levels (Level 5). Process evaluations were used to determine implementation status and direction (Levels 1-4 and 7), and a CM&E water quality sampling program was established to determine the overall impact on water quality (Level 7). Nationally, the CM&Es and other project evaluations were supplemented with statistically based socioeconomic phone surveys of participants and nonparticipants in the project areas. The combination of these three evaluations enabled national program managers and experts 1) to develop a list of recommendations concerning future agricultural nonpoint source control projects (Farm Bureau, 1992), 2) provide guidance to states to help them develop their own programs and 3) to determine the impact of RCWP on watershed level water quality.

Table 4. Hierarchy Levels as They Relate to Project Phase

Level	Component	Planning	Implementation	Evaluation
7	Behavior Changes		X	X
6	End Results		X	X
5	KASA Change		X	X
4	Reactions	X	X	X
3	Target Audience	X	X	
2	Activities	X	X	
1	Inputs	X	X	

A review of the Section 319 Program (which is more recent than RCWP) indicates that states and others have utilized the lessons learned from RCWP to design the next generation of agricultural Nonpoint Source Programs. The RCWP experience shows that ownership of the evaluation 1) increased the useability of the evaluation by the project implementation team, 2) increased the volume and improved the quality of material provided to the public and target audience during project implementation, and 3) decreased the response time between a concern or issue being raised and resolved with project-level data. The CM&E component proved invaluable in the overall evaluation of the RCWP Program.

Evaluation Barriers

In addition to some barriers already mentioned, there are a number of factors that may limit the scope, direction, and success of an evaluation. Leeds and others (1995) developed a list of factors that may influence an evaluation:

Organizational Politics
Organizational Structure

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Program Leadership
Professional Influences
Project/Program History
Economics
Social Patterns
Legal Guidelines

Each of these factors has implications for an evaluation. Examine the purpose of a project evaluation in the context of these factors. For example, is the evaluation going to be used to "get" someone (organizational politics)? Knowing which of these factors might have an impact on your evaluation will allow the evaluation team to modify data collection approaches and methods to account for it.

Closing

1) Evaluation should begin when the project/program begins and be part of the planning process.

2) Evaluations that are based on valid information are more useful in decision-making than those based on assumptions or opinions.

3) Evaluations that focus on average conditions and average operations may miss the importance of extremes.

4) Evaluation information that truly reflects the target audience results in better decision-making.

5) Hard data is generally more comfortable as a basis of making project decisions. However, hard data is usually more expensive and time-consuming to acquire.

6) Acceptance of evaluations is more likely when management is involved throughout the evaluation process.

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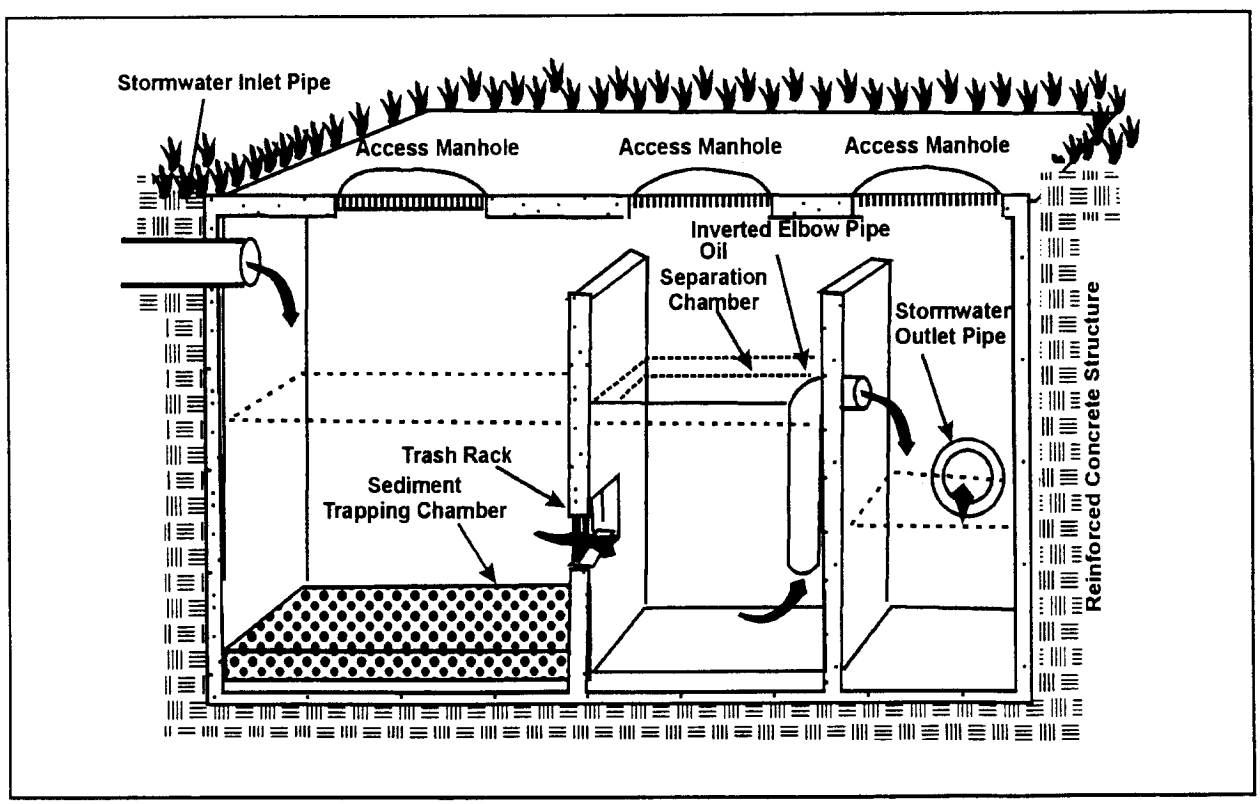
United States Environmental Protection Agency
Office of Water Washington, D.C.
EPA 832-F-99-029 September 1999

Storm Water Technology Fact Sheet
Water Quality Inlets

DESCRIPTION

Water quality inlets (WQIs), also commonly called oil/grit separators or oil/water separators, consist of a series of chambers that promote sedimentation of coarse materials and separation of free oil (as opposed to emulsified or dissolved oil) from storm water. Most WQIs also contain screens to help retain larger or floating debris, and many of the newer designs also include a coalescing unit that helps to promote oil/water separation. WQIs typically capture only the first portion of runoff for treatment and are generally used for pretreatment before discharging to other best management practices (BMPs).

A typical WQI, as shown in Figure 1, consists of a sedimentation chamber, an oil separation chamber, and a discharge chamber. The basic WQI design is often modified to improve performance. Possible



Source: Berg, 1991.

FIGURE 1 PROFILE OF A TYPICAL WATER QUALITY INLET

modifications include: an additional orifice and chamber that replace the inverted pipe elbow; the extension of the second chamber wall up to the top of the structure; or the addition of a diffusion device at the inlet. The diffusion device is intended to dissipate the velocity head and turbulence and distribute the flow more evenly over the entire cross-sectional area of the sedimentation chamber (API, 1990).

The addition of a coalescing unit to the WQI can dramatically increase its effectiveness in oil/water separation while also greatly reducing the size of the required unit. Coalescing units are made from oil-attracting materials, such as polypropylene or other materials. These units attract small oil droplets, which begin to concentrate until they are large enough to float to the surface and separate from the storm water. Without these units, the oil and grease particles must concentrate and separate naturally. This requires a much larger surface area; and therefore, units that do not use the coalescing process must be larger than units utilizing a coalescing unit.

WQIs can be purchased as pre-manufactured units (primarily oil/water separator tanks) or constructed on site. Suppliers of pre-manufactured units (e.g., Highland Tank and Manufacturing, Jay R. Smith Manufacturing, etc.) can also provide modifications of the typical design for special conditions.

APPLICABILITY

WQIs are widely used in the U.S. and can be adapted to all regions of the country. They are often used where land requirements and cost prohibit the use of larger BMP devices, such as ponds or wetlands. WQIs are also used to treat runoff prior to discharge to other BMPs.

Because of their ability to remove hydrocarbons, WQIs are typically located at sites with automotive-related contamination or at other sites that generate high hydrocarbon concentrations (MWCOG, 1993). For example, WQIs may be ideal for small, highly impervious areas, such as gas stations, loading areas, or parking areas (Schueler, 1992). Many WQIs, particularly those installed at industrial sites, serve the dual purpose of treating storm water runoff from contaminated areas, and serving as collection and treatment units for washdown processes or petroleum spills.

Higher residual hydrocarbon concentrations in trapped sediments cause maintenance and residual disposal costs associated with WQIs to be higher than those of other BMPs. Therefore, planners should carefully evaluate maintenance and residual disposal issues for the site before selecting a WQI. Possible alternatives to the WQI include sand filters, oil absorbent materials, and other innovative BMPs (e.g., Stormceptor System).

ADVANTAGES AND DISADVANTAGES

WQIs can effectively trap trash, debris, oil and grease, and other floatables that would otherwise be discharged to surface waters (Schueler, 1992). In addition, a properly designed and maintained WQI can serve as an effective BMP for reducing hydrocarbon contamination in receiving water sediments. While WQIs are effective in removing heavy sediments and floating oil and grease, they have demonstrated limited ability to separate dissolved or emulsified oil from runoff. WQIs are also not very effective at removing pollutants such as nutrients or metals, except where the metals removal is directly related to sediment removal.

Several major constraints can limit the effectiveness of WQIs. The first is the size of the drainage area. WQIs are generally recommended for drainage areas of 0.4 hectares (1 acre) or less (Berg, 1991, NVPDC, 1992). Construction costs often become prohibitive for larger drainage areas. However, because WQIs are primarily designed for specific industrial sites that have the potential for petroleum-contaminated process washdown, spills, and storm water runoff, sizing considerations are not usually a problem.

Sediment can also cause problems for WQIs. There are several reasons for this. First, high sediment loads can interfere with the ability of the WQI to effectively separate oil and grease from the runoff. Second, during periods of high flow, sediment residuals may be resuspended and released from the WQI to surface waters. A 1993 Metropolitan Washington Council of Governments (MWCOG)

long-term study evaluating the performance and effectiveness of more than 100 WQIs found that pollutants in the WQI sediments were similar to those pollutants found in downstream receiving water sediments (the tidal Anacostia River). This information suggests that downstream sediment contamination is linked to contaminated runoff and pass-through from WQIs (MWCOG, 1993). Third, WQI residuals accumulate quickly and require frequent removal. There is also some concern that because the collected residuals contain hydrocarbon by-products, the residuals may be considered too toxic for conventional landfill disposal. The 1993 MWCOG study found that the residuals from WQIs typically contain many priority pollutants, including polyaromatic hydrocarbons, trace metals, phthalates, phenol, toluene, and possibly methylene chloride (MWCOG, 1993). Based on these considerations, WQIs should not be implemented at sites that generate large amounts of sediment in the runoff unless the runoff has been pretreated to reduce the sediment loads to manageable levels.

WQIs are also limited by maintenance requirements. Maintenance of underground WQIs can be easily neglected because the WQI is often "out of sight and out of mind." Regular maintenance is essential to ensuring effective pollutant removal. As discussed above, lack of maintenance will often result in resuspension of settled pollutants.

Finally, WQIs generally provide limited hydraulic and residuals storage. Due to the limited storage, WQIs do not provide adequate storm water quantity control.

DESIGN CRITERIA

Prior to WQI design, the site should be evaluated to determine if another BMP would be more cost-effective in removing the pollutants of concern. WQIs should be used when no other BMP is feasible. The WQI should be constructed near a storm drain network so that flow can be easily diverted to the WQI for treatment (NVPDC, 1992). Any construction activities within the drainage area should be completed before installation of the WQI, and the drainage area should be revegetated so that the sediment loading to the WQI is minimized. Upstream sediment control measures should be implemented to decrease sediment loading.

WQIs are most effective for small drainage areas. Drainage areas of 0.4 hectares (1 acre) or less are often recommended. WQIs are typically used in an off-line configuration (i.e., portions of runoff are diverted to the WQI), but they can be used as on-line units (i.e., receive all runoff). Generally, off-line units are designed to handle the first 1.3 centimeters (0.5 inches) of runoff from the drainage areas. Upstream isolation/diversion structures can be used to divert the water to the off-line structure (Schueler, 1992). On-line units receive higher flows that will likely cause increased turbulence and resuspension of settled material, thereby reducing WQI performance.

As discussed above, oil/water separation tank units are often utilized in specific industrial areas, such as airport aprons, equipment washdown areas, or vehicle storage areas. In these instances, runoff from the area of concern will usually be diverted directly into the unit, while all other runoff is sent to the storm drain downstream from the oil/water separator. Oil/water separation tanks are often fitted with diffusion baffles at the inlets to prevent turbulent flow from entering the unit and resuspending settled pollutants.

WQIs are available as pre-manufactured units or can be cast in place. Reinforced concrete should be used to construct below-grade WQIs. The WQIs should be water tight to prevent possible ground water contamination.

Chamber Design

Structural loadings should be considered in the WQI design (Berg, 1991), particularly with respect to the sizing of the chambers. When the combined length of the first two chambers exceeds 4 meters (12 feet), the chambers are typically designed with the length of the first and second chamber being two-thirds and one-third of the combined length of the unit, respectively. Each of the chambers should have a separate manhole to provide access for cleaning and inspection.

The State of Maryland design standards indicate that the combined volume of the first and second chambers should be determined based on 1.1 cubic meters (40 cubic feet) per 0.04 hectares (0.10 acres) draining to the WQI. In Maryland, this is equivalent to capturing the first 0.33 centimeters (0.133 inches) of runoff from the contributing drainage area.

Permanent pools within the chambers help prevent the possibility of sediment resuspension. The first and second chambers should have permanent pools with depths of 1.2 meters (4 feet). If possible, the third chamber should also contain a permanent pool (NVPDC, 1992).

The first and second chambers are generally connected by an opening covered by a trash rack, a PVC pipe, or other suitable material pipe (Berg, 1991). If a pipe is used, it should also be covered by a trash rack or screen. The opening or pipe between the first and second chambers should be designed to pass the design storm without surcharging the first chamber (Berg, 1991). The design storm will vary depending on geographical location and is generally defined by local regulations.

In the standard WQI, an inverted elbow is installed between the second and third chamber. The elbow should extend a minimum of 1 meter (3 feet) into the second chamber's permanent pool. Because oil will naturally separate from, and float on top of, the water, water will be forced through the submerged elbow and into the third chamber while oil will be retained in the second chamber (NVPDC, 1992). The depth of the elbow into the permanent pool should should be. The size of the elbow or the number of elbows can be adjusted to accommodate the design flow and prevent discharge of accumulated oil(Berg, 1991).

Pre-manufactured oil/water separation tanks do not usually follow the separated-chamber design; instead, these units often rely on baffle units to separate the different removal process. Particulates are thus retained near the inlet to the tank, while oil/water separation takes place closer to the tank outlet.

PERFORMANCE

WQIs are primarily utilized to remove sediments from storm water runoff. Grit and sediments are partially removed by gravity settling within the first two chambers. A WQI with a detention time of 1 hour may expect to have 20 to 40 percent removal of sediments. Hydrocarbons associated with the accumulated sediments are also often removed from the runoff through this process. The WQI achieves slight, if any, removal of nutrients, metals and organic pollutants other than free petroleum products (Schueler, 1992).

The 1993 MWCOG study discussed above found that an average of less than 5 centimeters (2 inches) of sediments (mostly coarse-grained grit and organic matter) were trapped in the WQIs. Hydrocarbon and total organic carbon (TOC) concentrations of the sediments averaged 8,150 and 53,900 milligrams per kilogram, respectively. The mean hydrocarbon concentration in the WQI water column was 10 milligrams per liter. The study also indicated that sediment accumulation did not increase over time, suggesting that the sediments become re-suspended during storm events. The authors concluded that although the WQI effectively separates oil and grease from water, re-suspension of the settled matter appears to limit removal efficiencies. Actual removal only occurs when the residuals are removed from the WQI (Schueler 1992).

A 1990 report by API found that the efficiency of oil and water separation in a WQI is inversely proportional to the ratio of the discharge rate to the unit's surface area. Due to the small capacity of the WQI, the discharge rate is typically very high and the detention time is very short. For example, the MWCOG study found that the average detention time in a WQI is less than 0.5 hour. This can result in minimal pollutant settling (API, 1990). However, the addition of coalescing units in many current WQI units may increase oil/water separation efficiency. Most coalescing units are designed to achieve a specific outlet concentration of oil and grease (for example, 10-15 parts per million oil and grease).

OPERATION AND MAINTENANCE

The key to the performance of WQIs is maintenance. When properly maintained, WQIs should experience very few separation, clogging, or structural problems.

Basic maintenance should consist of regularly checking and cleaning out the sediment that has accumulated in the WQI. A lack of regular clean-outs can lead to the resuspension of collected sediments; therefore, WQIs should be inspected after every storm event to determine if maintenance is required. At a minimum, each WQI should be cleaned at the beginning of each season (Berg, 1991). The required maintenance will be site-specific due to variations in sediment and hydrocarbon loading. Maintenance should include clean out, disposal of the sediments, and removal of trash and debris. The clean out and disposal techniques should be environmentally acceptable and in accordance with local regulations. Since WQI residuals contain hydrocarbon by-products, they may require disposal as hazardous waste. Many WQI owners coordinate with waste haulers to collect and dispose of these residuals. Since WQIs can be relatively deep, they may be designated as confined spaces. Caution should be exercised to comply with confined space entry safety regulations if it is required.

Oil/water separator tank units can be fitted with sensing units that will indicate when the units need to be cleaned. Because most of oil/water separator tank units are designed for specific industrial applications, their maintenance schedule should be closely tied to the industrial process schedule. However, these units should also be inspected after rain events.

COSTS

The construction costs for WQIs will vary greatly depending on their size and depth. The construction costs (in 1993 dollars) for cast-in-place WQIs range from $5,000 to $16,000, with the average WQI costing around $8,500 (Schueler, 1992). For the basic design and construction of WQIs, the pre-manufactured units are generally less expensive than those that are cast in place (Berg, 1991).

Maintenance costs will also vary greatly depending on the size of the drainage area, the amount of the residuals collected, and the clean out and disposal methods available (Schueler, 1992). The cost of residuals removal, analysis, and disposal can be a major maintenance expense, particularly if the residuals are toxic and are not suitable for disposal in a conventional landfill.

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United States Environmental Protection Agency
Office of Water Washington, D.C.
EPA 832-F-99-007 September 1999

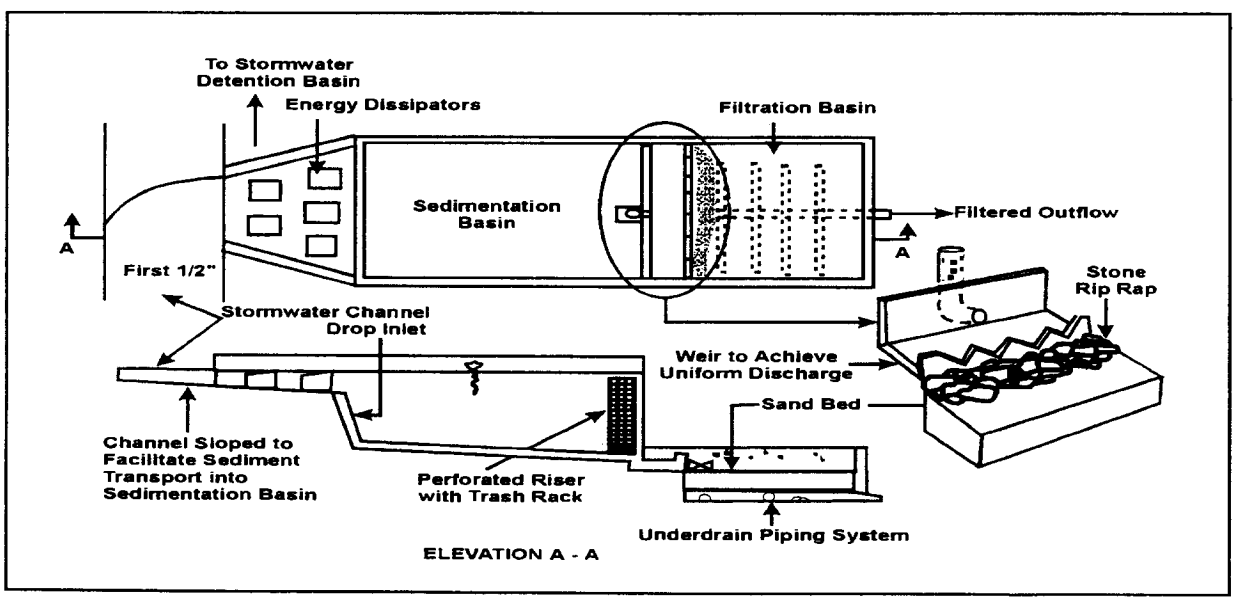
Storm Water Technology Fact Sheet
Sand Filters

DESCRIPTION

Sand filters have proven effective in removing several common pollutants from storm water runoff. Sand filters generally control storm water quality, providing very limited flow rate control.

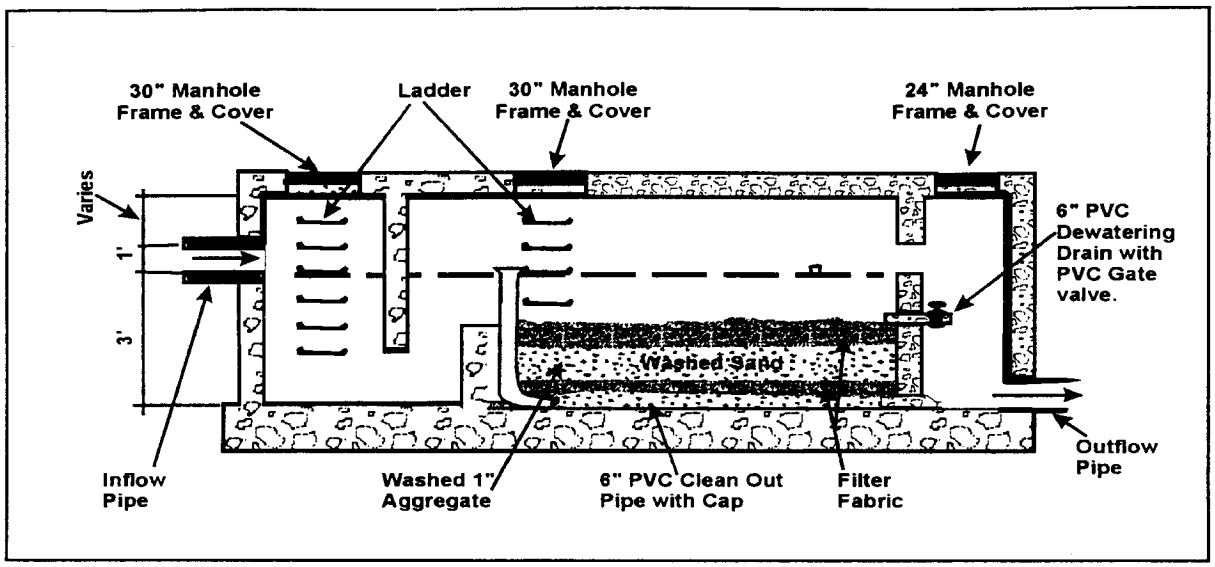
A typical sand filter system consists of two or three chambers or basins. The first is the sedimentation chamber, which removes floatables and heavy sediments. The second is the filtration chamber, which removes additional pollutants by filtering the runoff through a sand bed. The third is the discharge chamber. The treated filtrate normally is then discharged through an underdrain system either to a storm drainage system or directly to surface waters. Sand filters take up little space and can be used on highly developed sites and sites with steep slopes. They can be added to retrofit existing sites. Sand filters are able to achieve high removal efficiencies for sediment, biochemical oxygen demand (BOD), and fecal coliform bacteria. Total metal removal, however, is moderate, and nutrient removal is often low.

There are three main sand filter designs currently in common use: the Austin sand filter (Figure 1); the Washington, D.C., sand filter (Figure 2); and the



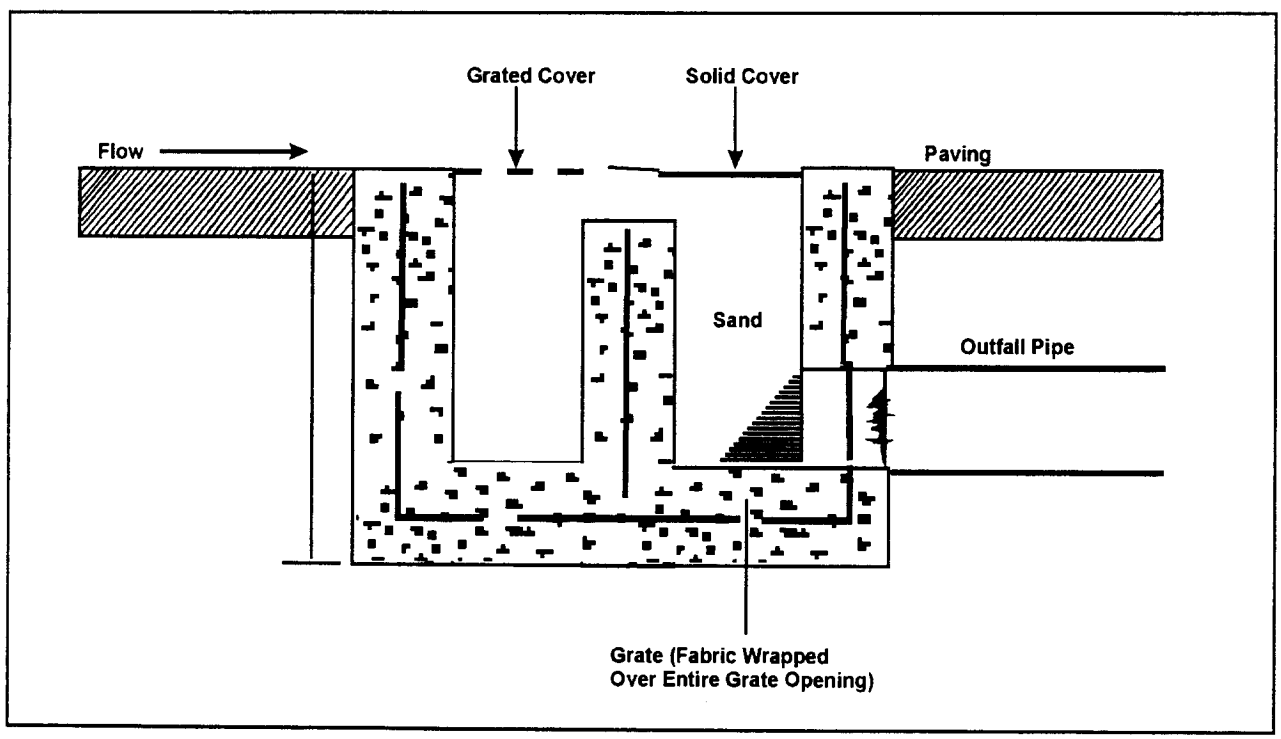
Source: Schueler, 1992.

FIGURE 1 TYPICAL AUSTIN SAND FILTER DESIGN



Source: Troung, 1989.

FIGURE 2 TYPICAL WASHINGTON, D.C. SAND FILTER DESIGN



Source: Shaver, 1991.

FIGURE 3 TYPICAL DELAWARE SAND FILTER DESIGN

Delaware sand filter (Figure 3). The primary differences among these designs are location (i.e., above or below ground), the drainage area served, their filter surface areas, their land requirements, and the quantity of runoff they treat.

Modifications that may improve sand filter design and performance are being tested. One modification is the addition of a peat layer in the filtration chamber. The addition of peat to the sand

filter may increase microbial growth within the sand filter and improve metals and nutrient removal rates.

APPLICABILITY

Sand filters are intended primarily for water quality enhancement. In general, sand filters are preferred over infiltration practices, such as infiltration trenches, when contamination of groundwater with conventional pollutants - BOD, suspended solids, and fecal coliform - is of concern. This usually occurs in areas where underlying soils alone cannot treat runoff adequately - or ground water tables are high. In most cases, sand filters can be constructed with impermeable basin or chamber bottoms, which help to collect, treat, and release runoff to a storm drainage system or directly to surface water with no contact between contaminated runoff and groundwater.

The selection of a sand filter design depends largely on the drainage area's characteristics. For example, the Washington, D.C., and Delaware sand filter systems are well suited for highly impervious areas where land available for structural controls is limited, since both are installed underground. They are often used to treat runoff from parking lots, driveways, loading docks, service stations, garages, airport runways/taxiways, and storage yards. The Austin sand filtration system is more suited for large drainage areas that have both impervious and pervious surfaces. This system is located at grade and is often used at transportation facilities, in large parking areas, and in commercial developments.

In general, all three types of sand filters can be used as alternatives for water quality inlets. They are more frequently used to treat runoff contaminated with oil and grease from drainage areas with heavy vehicle usage. In regions where evaporation exceeds rainfall and a wet pond would be unlikely to maintain the required permanent pool, the Austin sand filtration system can be used.

ADVANTAGES AND DISADVANTAGES

Sand filters can be highly effective storm water best management practices (BMPs). All three types of sand filters achieve high removal rates for sediment, BOD, and fecal coliform bacteria. The filter media is periodically removed from the filter unit, thus also permanently removing trapped contaminants. Waste media from the filters does not appear to be toxic and is environmentally safe for landfill disposal. If they are designed with an impermeable basin liner, sand filters can also reduce the potential for groundwater contamination. Finally sand filters also generally require less land than other BMPs, such as ponds or wetlands.

The size and characteristics of the drainage area, as well as the pollutant loading, will greatly influence the effectiveness of the sand filter system. For example, sand filters may be of limited value in some applications because of they are designed to handle runoff from relatively small drainage areas and they have low nutrient removal and metal removal capabilities. In these cases, other BMPs, such as wet ponds, may be less costly and/or more effective. The system also requires routine maintenance to prevent sediment from clogging the filter. In some cases, filter media may need to be replaced 3 to 5 years. Lastly, sand filters generally do not control storm water flow, and consequently, they do not prevent downstream stream bank and channel erosion.

Climatic conditions may also limit the filter's performance. For example, it is not yet known how well sand filters will operate in colder climates or in freezing conditions.

DESIGN CRITERIA

Typically the Austin sand filter system is designed to handle runoff from drainage areas up to 20 hectares (50 acres). The collected runoff is first diverted to the sedimentation basin, where heavy sediments and floatables are removed. There are two designs for the sedimentation basin: the full sedimentation system, as shown in Figure 1; and a partial sedimentation system, where only the initial flow is diverted. Both systems are located off-line and are designed to collect and treat the first 1.3 centimeters (0.5 inches) of runoff. The partial system has the capacity to hold only a portion (at least 20 percent) of the first flush volume in the sedimentation basin, whereas the full system captures and holds the entire flow volume.

Equations used to determine the sedimentation basin surface areas (As) in square and meters acres are shown in Table 1.

TABLE 1 SURFACE AREA EQUATION FOR AUSTIN SAND FILTER SYSTEM

Partial Sedimentation	Full Sedimentation
As=(AD)(H)/(1/Ds-1/10)	As=(AD)(H)/10
Af=(AD)(H)/10	Af=(AD)(H)/18

Note: Designed to collect and treat 0.5 inches of runoff.
Ds (feet)=depth of the sedimentation basin.
H (feet)=depth of rainfall, 0.042ft (0.5 in).
AD(acres)=impervious and pervious areas that provide contributing drainage.

Source: Galli, 1990.

Flow is conveyed from the sedimentation basin, through a perforated riser, a gabion wall, or a berm, to the filtration basin. The filtration basin consists of a 45-centimeter (18-inch) layer of sand particles 0.05 to 0.10 centimeters (0.02 to 0.04 inches) in diameter that may be underlain by a gravel layer. Equations used to determine the surface areas (Af) in acres are also shown in Table 1. The filtrate is discharged from the filtration basin through underdrain piping 10 to 15 centimeters (4 to 6 inches) in diameter with 1-centimeter (0.4 inch) perforations. Filter fabric is placed around the underdrain piping to prevent sand and other particulates from being discharged.

Typically, the Washington, D.C., sand filter system is designed to handle runoff from completely impervious drainage areas of 0.4 hectares (1 acre) or less. The system, as shown in Figure 2, consists of three underground chambers: a sedimentation chamber, a filtration chamber, and a discharge chamber. The sand filter system is designed to accept the first 1.3 centimeters (0.5 inches) of runoff. Coarse sediments and floatables are removed from the runoff within the sedimentation chamber. Runoff is discharged from the sedimentation chamber through a submerged weir, into the filtration chamber, which consists of a combination of sand and gravel layers totaling 1 meter (3 feet) in depth with underdrain piping wrapped in filter fabric. The underdrain system collects the filtered water and discharges it to the third chamber, where the water is collected and discharged to a storm water channel or sewer system. An overflow weir is located between the second and third chambers to bypass excess flow. The Washington, D.C., sand filter is often constructed on-line, but can be constructed off-line. When the system is off-line, the overflow between the second and third chambers is not included.

The Delaware sand filter, shown in Figure 3, is similar to the Washington, D.C., sand filter in that both utilize underground concrete vaults. However, the Delaware sand filter has only two chambers: a sedimentation chamber and a filtration chamber. A 2.5-centimeter (1 inch) design storm was selected for sizing the sedimentation basin because it is representative of large storm events: in Delaware, 92 percent of all storms are less than 2.5 centimeters (1 inch) in depth. Runoff enters the sedimentation chamber through a grated cover and then overflows into the filtration chamber, which contains a sand layer 45 centimeters (18 inches) in depth. Gravel is not normally used in the filtration chamber although the filter can be modified to include it. Typical systems are designed to handle runoff from drainage areas of 2 hectares (5 acres) or less. A major advantage of the Delaware sand filter is its shallow structure depth of only 76 centimeters (30 inches), which reduces construction and maintenance costs.

Proper design and maintenance are also critical factors in maintaining the operating life of any filter system. The life of the filter media may be increased by a number of methods, including:

- Stabilizing the drainage area so that sediment loadings in the runoff are minimized.
- Providing adequate storm water detention times to enhance sedimentation and filtration.
- Inspecting and maintaining the sand filter frequently enough to ensure proper operation.

PERFORMANCE

Sand filters are currently in use in Delaware, Maryland, Florida, Texas, Virginia, and Washington, D.C. Studies on the systems' pollutant removal efficiencies are currently being performed in Washington, D.C., and Austin, TX. Additional evaluations are needed to evaluate alternative sand filter designs and media. Sand filters remove particulates in both the sedimentation and the filtration chambers. The City of Austin has estimated their systems' pollutant removal efficiencies based on preliminary findings of their storm water monitoring program (Austin, 1988). The estimates shown in Table 2 are average values for various sand filters serving drainage areas of several different sizes. As shown in Table 2, no removal of nitrate was observed. No other dissolved pollutants were monitored. Additional monitoring is currently being performed by the City of Austin to supplement the preliminary estimates.

TABLE 2 TYPICAL POLLUTANT REMOVAL EFFICIENCY

Pollutant	Percent Removal
Fecal Coliform	76
Biochemical Oxygen Demand (BOD)	70
Total Suspended Solids (TSS)	70
Total Organic Carbon (TOC)	48
Total Nitrogen (TN)	21
Total Kjeldahl Nitrogen (TKN)	46
Nitrate as Nitrogen (NO_3-N)	0
Total Phosphorus (TP)	33
Iron (Fe)	45
Lead (Pb)	45
Zinc (Zn)	45

Source: Galli, 1990

OPERATION AND MAINTENANCE

All filter system designs must provide adequate access to the filter for inspection and maintenance. The sand filters should be inspected after all storm events to verify that they are working as intended. Since the Washington, D.C., and Austin sand filter systems can be deep, they may be designated as confined spaces and require compliance with confined space entry safety procedures.

Typically, sand filters begin to experience clogging problems within 3 to 5 years (NVPDC, 1992). Accumulated trash, paper and debris should be removed from the sand filters every 6 months or as necessary to keep the filter clean. A record should be kept of the dewatering times for all sand filters to determine if maintenance is necessary. Corrective maintenance of the filtration chamber includes removal and replacement of the top layers of sand, gravel and/or filter fabric that has become clogged. The removed media may usually be disposed in a landfill. The City of Austin tests their waste media before disposal. Results thus far indicate that the waste media is not toxic and can be safely landfilled (Schueler, 1992). Sand filter systems may also require the periodic removal of vegetative growth.

COSTS

The construction cost for an Austin sand filtration system is approximately $18,500 (1997 dollars) for a 0.4 hectare- (1 acre-) drainage area. The cost per hectare decreases with increasing drainage area. The cost for precast Washington, D.C. sand filters, with drainage areas of less than 0.4 hectares (1 acre), ranges between $6,600 and $11,000 (1997 dollars). This is considerably less than the cost for the same size cast-in-place system. Costs for the Delaware sand filter are similar to that of the D.C. system, with the exception of the lower excavation costs due to the Delaware filters' shallowness.

Annual costs for maintaining sand filter systems average about 5 percent of the initial construction cost (Schueler, 1992). Media is replaced as needed. Currently the sand is being replaced in the D.C. filter systems about every 2 years. The cost to replace the gravel layer, filter fabric and top portion of the sand for D.C. sand filters is approximately

$1,700 (1997 dollars). Improvements in Washington, D.C.'s maintenance procedures may extend the life of the filter media and reduce the overall maintenance costs.

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United States Environmental Protection Agency
Office of Water Washington, D.C.
EPA 832-F-99-011 September 1999

EPA

Storm Water O&M Fact Sheet Catch Basin Cleaning

DESCRIPTION

Catch basins are chambers or sumps, usually built at the curb line, which allow surface water runoff to enter the storm water conveyance system. Many catch basins have a low area below the invert of the outlet pipe intended to retain coarse sediment. By trapping sediment, the catch basin prevents solids from clogging the storm sewer and being washed into receiving waters. Catch basins must be cleaned periodically to maintain their ability to trap sediment, and consequently their ability to prevent flooding. The removal of sediment, decaying debris, and highly polluted water from catch basins has aesthetic and water quality benefits, including reducing foul odors, reducing suspended solids, and reducing the load of oxygen-demanding substances that reach receiving waters.

APPLICABILITY

Catch basin cleaning should be performed at any facility that has an on-site storm sewer system that includes catch basins and manholes.

Although catch basin cleaning is easily implemented, it is often overlooked in an overall storm water management plan. In addition, many of the catch basin cleaning programs that have been implemented focus only on removal of debris from grate openings; full implementation of the catch basin cleaning BMP should also include removal of debris from the catch basin itself.

ADVANTAGES AND DISADVANTAGES

Catch basin cleaning is an efficient and cost-effective method for preventing the transport of sediment and pollutants to receiving water bodies. This improves both the aesthetics and the quality of the receiving water body.

Limitations associated with cleaning catch basins include:

- Catch basin debris usually contains appreciable amounts of water and offensive organic material which must be properly disposed.
- Catch basins may be difficult to clean in areas with poor accessibility and in areas with traffic congestion and parking problems.
- Cleaning is difficult during the winter when snow and ice are present.

Sediment and debris removed from catch basins can potentially be classified as hazardous waste. As a result, the materials must be disposed in a proper manner to avoid negative environmental impacts.

PERFORMANCE

Based on current data, it is not possible to quantify the water quality benefits to receiving waters resulting from catch basin cleaning. The rate at which catch basins fill with debris, as well as the total amount of material which can be removed by different frequencies of cleaning, are highly variable and cannot be readily predicted. Past studies have estimated that typical catch basins retain up to 57 percent of coarse solids and 17 percent of equivalent biological oxygen demand (BOD).

In addition, data collected as part of a Nationwide Urban Runoff Program (NURP) project in Castro Valley Creek, California, indicated that catch basins, cleaned on an average of once every year and a half, contained approximately 60 pounds of material each at the time of the cleaning.

OPERATION AND MAINTENANCE

Catch basins should be inspected at least annually to determine if they need to be cleaned. Typically, a catch basin should be cleaned if the depth of deposits is greater than or equal to one-third the depth from the basin to the invert of the lowest pipe or opening into or out of the basin. If a catch basin significantly exceeds the one-third depth standard during the annual inspection, then it should be cleaned more frequently. If woody debris or trash accumulates in a catch basin, then it should be cleaned on at least a weekly basis.

Catch basins can be cleaned either manually or by specially designed equipment. This equipment may include bucket loaders and vacuum pumps. Material removed from catch basins is usually disposed in conventional landfills. Before any materials can be disposed, it is necessary to perform a detailed chemical analysis to determine if the materials meet the EPA criteria for hazardous waste. This will help determine how the materials should be stored, treated, and disposed.

COSTS

Catch basin cleaning costs will vary depending upon the method used, the required cleaning frequency, the amount of debris removed, and debris disposal costs.

Cleaning costs for catch basins were estimated in three NURP program studies (Midwest Research Institute, 1982). These estimates are summarized in Table 1.

In communities equipped with vacuum street sweepers, a cleaning cost of $8 per basin cleaned is recommended for budgetary purposes (Southeastern Wisconsin Regional Planning Commission, 1991.) Cleaning catch basins manually costs approximately twice as much as cleaning the basins with a vacuum attached to a sweeper. Therefore, a cost estimate of $16 per catch basin cleaned may be used for manual cleaning. It should be noted that costs vary depending on local market conditions.

TABLE 1 CLEANING COST PER CATCH BASIN

Location	Method	Cost
Castro Valley, CA	Vacuum attached to street sweeper	$7.70
Salt Lake County, UT	Vacuum attached to street sweeper	$10.30
Winston-Salem, NC	Vacuum attached to street sweeper	$6.30

Source: MRI, 1982.

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HARNESSING THE POWER OF MICROSOFT ACCESS FOR THE MANAGEMENT OF NPDES PERMIT COMPLIANCE DATA IN A MULTI-PERMITTEE SCENARIO

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ABSTRACT

Full implementation of the United States Environmental Protection Agency's (EPA) NPDES MS4 permit program imposes extensive compliance and reporting requirements on municipalities. Typical requirements include development, revision, and implementation of a comprehensive Storm Water Management Program (SWMP). An overall Annual Report needs to be submitted to EPA, which qualitatively and quantitatively describes the specific task accomplishments and compliance status of each permittee.

EPA is sued a MS4 Permit to Pinellas County and 22 co-permitees, effective November 1,1997. Pinellas County contracted PBS&J to develop a user-friendly and comprehensive data management system to collect, compile and summarize permit compliance data from all the 23 co-permittees and generate the annual summary report for submission to EPA.

In order to facilitate the collection, analysis and compilation of a vast amount of data from these permittees, Pinellas County contracted PBS&J to develop a user-friendly and comprehensive data management system to perform this challenging task. Though this task appears to be simple in concept, implementation was complicated due to these factors: the portions of permit applicable for each co-permittee vary significantly; the type of data to be collected and reported varied for each co-permittee; no common data collection, storage or analysis methodology existed among the permittees at the time of this project; and the GIS/Database system of the County is not used by several of the co-permittees.

PBS&J successfully accomplished this task through a team of MS4 Permit Experts, GIS Analysts, and Database Designers in conjunction with the staff from all the co-permittees. The Pinellas County NPDES Permit Tracking System was developed as a flexible stand-alone application in Microsoft Access, with a simple and intuitive graphical user interface that even an inexperienced computer user could use with minimal training. Each co-permitee department has the ability to specify their own performance measures without affecting the performance of other departments or co-permittees. Generation of annual summary reports is as simple as a click of a button. The database was designed for use in both a multi-user network environment and a single-user desktop setting, and can be expanded to incorporate spatial intelligence using GIS. This project is a demonstration of the power of applying the new information technology tools in conjunction with specialized functional knowledge of MS4 permits to simplify an otherwise daunting task of tracking NPDES permit compliance activities.

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INTRODUCTION

In response to the need for comprehensive National Pollutant Discharge Elimination System (NPDES) requirements for discharge of storm water, Congress amended the Clean Water Act in 1987 to require the U.S. EPA to establish phased NPDES requirements for storm water discharges. To implement these requirements, EPA published the initial permit application requirements in November 1990 for certain categories of storm water discharges associated with industrial activity and discharges from municipal separate storm sewer systems serving populations of 100,000 or more. Municipal categories were classified as medium or large if they serve populations greater than 100,000 or more and 250,000 or more respectively. Applications for these permits were submitted by large and medium municipalities by November 1992 and May 1993 respectively. Many permits have been issued to date throughout the country.

Full implementation of the NPDES MS4 permit program has imposed extensive compliance and reporting requirements on municipalities throughout the country. Requirements for a typical MS4 permit include the development, revision, and implementation of a comprehensive Storm Water Management Program (SWMP) including pollution prevention measures, treatment or removal techniques, storm water monitoring, use of legal authority and other appropriate means to control the quality of storm water discharge from the MS4.

Pinellas County and 22 co-permittees were issued an MS4 permit, which became effective November 1,1997. Part V of the MS4 Permit requires submission of an overall Annual Report at the end of each permit year, which describes in both narrative and quantitative terms, the task accomplishments and compliance status of each permittee with reference to permit requirements. Summarizing the permit activities and preparing an annual summary report is a challenging task even in a single permittee scenario. With the need to summarize and report the activities of 23 different permittees, the complexity of this task increased many folds. Some of the factors that made this data compilation task more challenging were:

The applicable permit parts were different for each co-permitee

The type of data to be collected for each co-permittee varied in terms of what is reported and how the data was tracked

There was no common approach for data collection, storage and analysis among the co-permittees

The County's preferred database system was not used by other co-permittees

The resources available for the MS4 permit compliance activities were significantly different among the co-permittees, and

A suitable application was to be developed on a short notice due to the time constraints for submission of annual report

The NPDES project managers for Pinellas county and the co-permittees were knowledgeable of the complexity of this task and contracted PBS&J to develop a simple, straight forward method to collect, analyze, and summarize the data related to the MS4 permit compliance, and to automatically generate the Annual Report from the collected data. The County staff also had a vision to expand the NPDES data management system in the future to automate data transfer and data exchange

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operations between this system and the County's Maxim0 Work Management System, Oracle system and the GIS system.

Technical Approach

The overall objective of this project was study the MS4 permit requirements and develop a comprehensive and user-friendly data management system to collect, compile and summarize the permit compliance activities from all the permittees and their departments. The product of this data management system is the Annual Summary Report to be submitted to the EPA. In order to best meet the requirements of the County and the Co-Permittees, PBS&J developed a two-phased approach for this project.

Phase I - Application Development in MS Access: In this phase a custom relational database application was developed in Microsoft Access with standard data input screens for all users and a standard report module for all users to automatically generate Annual Summary Reports from the input data.

Phase II - Automated Input Routines for the County: This is a proposed phase in which a Bi-Directional Interface would be developed between the County's database systems (Maximo, Oracle, GIS) and the NPDES Application developed in Phase I, to facilitate seamless data exchange and eliminate duplicate data entry operations.

This paper discusses the design and development of the NPDES Permit Management System implemented to meet the Phase-1 requirements discussed above.

Relational Database Design Fundamentals

We are constantly dealing with different types of ***data*** in our daily life. Data is everywhere, but data is not ***information.*** Information is data that is organized in a meaningful form with a well-defined structure. Good data management provides the structure necessary for transforming a maze of data into information. A relational database is composed of a number of data tables related to each other through common fields. This facilitates in searching for information across several tables efficiently, economically, and accurately. This makes the data more accessible, easy to maintain, update and use. A relational database management system (RDBMS) is a collection of programs that enables users to create and maintain a relational database (Simpson and Olson, 1997).

Prior to the information revolution, such database management needed a high level of computer knowledge and programming skills. Engineers seldom had such level of skills and therefore had to resort to traditional data management methods. The advent of Windows-based database software with simple Graphical User Interfaces (GUI) virtually eliminated the need of programming knowledge to harness the power of database systems. Database systems also facilitate the implementation of a security protocol for data access.

We all have our own data management systems in place (predominantly spreadsheets). Though they may seem to work fine at the individual level or within small groups, they are likely to be

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corrupt the data when multiple users start managing the same data. We end up in situations where we have multiple copies of the same information and have difficulty in identifying the latest and most accurate data. In contrast to spreadsheets, relational database systems are easy-to-use tools for setting up a good data management practice. Using a RDBMS, we can quickly create queries to perform tasks that would have been very complicated to do with spreadsheets and generate a high quality report to summarize your analysis. The number of records (rows) one can have in a file is also a major limitation of spreadsheets. Lotus 1-2-3 (Release 5) allows 8 192 rows and Excel 97 allows 65,536 rows. So if we have large data sets like historical rainfall data or lake levels data or canal stage data, we now have the capability of using databases. We at PBS&J successfully used Access for data sets up to 10 million records.

Microsoft Access is the most popular desktop database in the market today. It is a part of the MS Office Professional Edition. It is easy to use yet powerful enough to dramatically improve our traditional data management systems. Due to its popularity, simplicity of use and its capabilities, MS Access was chosen as the RDBMS environment for developing the NPDES Permit Tracking System.

Setting up the Database Design

Any RDBMS is only as good as the design of the underlying tables and their relationships. In order to develop a good database design, it is essential to have a development team that has sound, functional knowledge of the problem as well as good database software designers. It is also imperative to discuss the needs of the clients in detail and get their approval prior to embarking on the design process (Elmasri and Navathe, 1994).

PBS&J assembled a team of MS4 Permit Experts, Storm Water Engineers, GIS Analysts, and Database Designers in order to develop the database design for the NPDES Permit Management System. Meetings were held with the responsible staff from all the 23 co-permittees. The MS4 permit experts studied the permit in great detail to understand the key items and requirements. A typical MS4 permit consists of nine (9) major program elements:

Structural Controls and Stormwater Collection System Operation
Areas of New Development and Significant Redevelopment
Roadways
Flood Control Projects
Municipal Waste Treatment, Storage or disposal facilities not Covered by an NPDES Storm Water Permit
Pesticide, Herbicide, and Fertilizer Application
Illicit Discharges and Improper Disposal
Industrial and High Risk Runoff
Construction Site Runoff

Each of the program elements requires a set of tasks to be performed in order to achieve compliance. Each such task was assigned a task number and the activities required to be performed under each task to achieve compliance were outlined by the MS4 experts. These activities were designated as performance measures and the activities performed by each co-permitee under each performance measure were to be summarized accordingly. The performance measures were grouped

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under four major categories namely date, inventory, project and compliance status. Each of these groups is briefly discussed below.

Date Dependent Activities: Date dependent activities are those actions that need to be completed by a specific date provided within the NPDES permit. These are actions such as completing a report by the end of the first permit year or implementing a specific program within 24 months of the effective date of the permit.

Inventory Driven Activities: Inventory driven activities are those activities for which a count will be provided in the annual report to EPA. These are activities such as screening a percentage of your total outfalls for potential pollutants or recording the number of public education activities provided. The system continues to keep track of all data entries so that at any point in time the user can see the total number reported to date prior to entering new records.

Project Related Activities: Project related activities are those activities where the compliance action is tied directly to individual projects and is managed on a project by project basis. These are such activities as keeping track of new development activities or making sure new projects comply with applicable best management practices as outlined in the NPDES permit document. All individual data entries are maintained within the system and the annual report is designed to generate a summary table showing how many projects were reviewed and found to be in compliance and how many were reviewed and found not to be in compliance.

Compliance Status Activities: Compliance status check activities are those activities for which a simply yes or no answer is sufficient to satisfy the action required by the NPDES permit. These are such activities as maintaining internal records or form a committee. The status of compliance may change throughout the permit year. As the compliance status changes and that information is recorded within the system, all individual entries will be maintained, however, the last change will be the status that is recorded within the annual report.

MS4 permit requirements were broken down into simple tasks. Each task was provided with a list of suggested performance measures, which would help in ensuring permit compliance. This formed the basis for the development of a database system in Microsoft Access environment to track all the compliance actions. The database was designed for use in both a multi-user network environment and a single-user desktop setting. The relationship between each of these database elements is depicted in Figure 1.

Development of Graphical User Interface

In order for the RDBMS to be utilized effectively by Pinellas County and the co-permittees it had to be created with an Intuitive and user friendly graphical interface. It was the goal of this program to ensure ease of use due to the varied computer experience of many of the co-permitees To accomplish ease of use, one of the most important considerations is the logical flow of information review and input. Additionally, no one form can contain more information than a typical user can digest quickly.

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Additional considerations included the ability to incorporate security measures. Initial login screens were developed to allow the user to choose their appropriate group and input a password before having access to any data entry screens. These forms were developed with consistent look and feel as the data entry forms to give the user a consistent interface from which to work.

Finally, the creation of the annual report documents needed to be accessed through these forms. The users needed to be provided flexibility to modify the reporting period. Custom coding was implemented in order to provide this flexibility while still preventing the user to modify the report format and design. Sample log-in, data entry and report creation screens are presented in Figure 2 to illustrate the easy-to-use visual interface developed for the NPDES Permit Management System.

Database Usage

Database usage follows a logical progression. The user logs into the database by choosing their appropriate group and entering the approved password. Based upon who logs in, the system automatically sub-sets the permit requirements based upon guidelines established at each participating municipality. The user can immediately upon entering the system review these overall requirements for which they have responsibility, generate summary report documents, or begin to add or edit data. If the user chooses to add or edit data they are provided the option to choose the particular required action for which they would like to enter data. Once the compliance data has been entered into the system, many different report formats are available for permit managers to review the data at various levels of summary. Certain users have additional access to modify the performance measures for required actions in order to better accommodate their business practices. The user navigation flow chart is documented in Figure 3.

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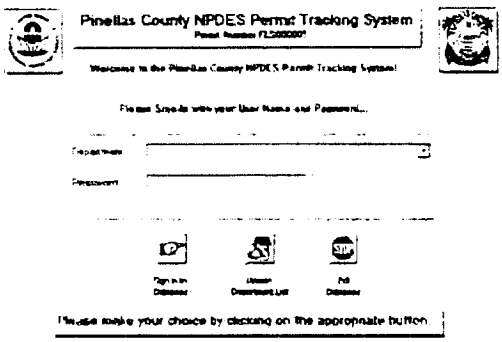
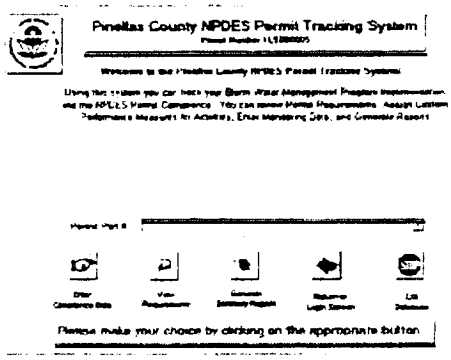
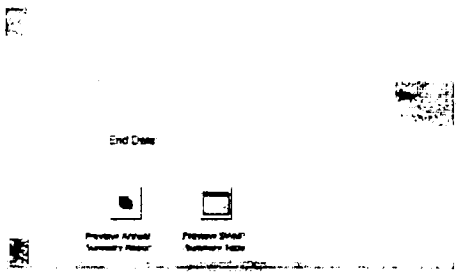
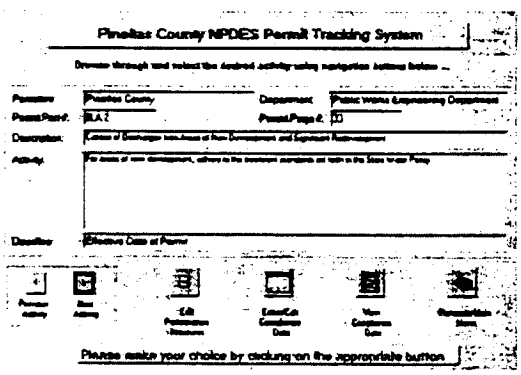
Main Program Entry Screen



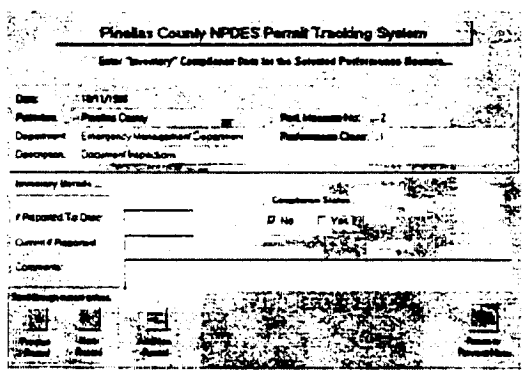




Permit Task Selection Screen



Example Data Entry Screen



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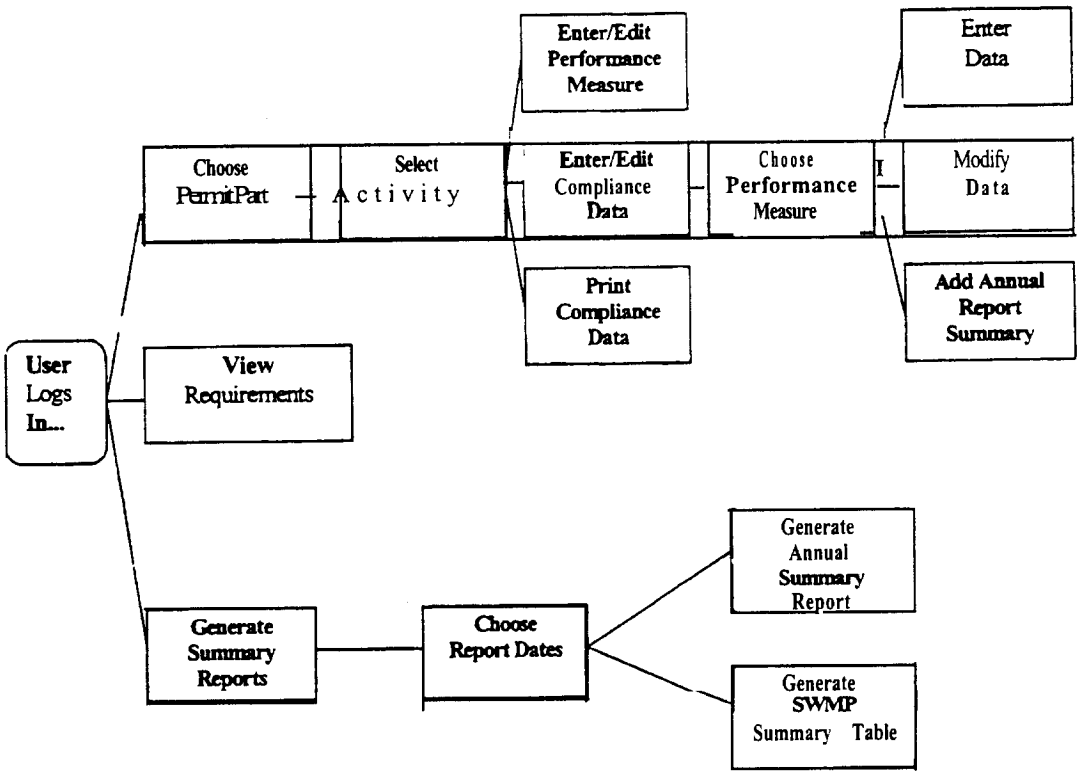

CONCLUSIONS

This database system is emerging as a valuable tool for the County's NPDES Program Coordinator who has the onerous task ofpreparing the Annual Permit Compliance Summary Report. The database can be expanded in the future to incorporate spatial intelligence using GIS or can be integrated with other County databases. This project is a demonstration of the power of applying state-of the art information technology tools in conjunction with specialized functional knowledge of MS4 permits to simplify an otherwise daunting task of tracking permit compliance activities of 23 different co-permittees and their departments. It is imperative for the civil engineers of the next millennium to be aware of the new developments in information technology and be able to harness the power of the new software tools. This will result in developing innovative and more efficient solutions to many of our project tasks.

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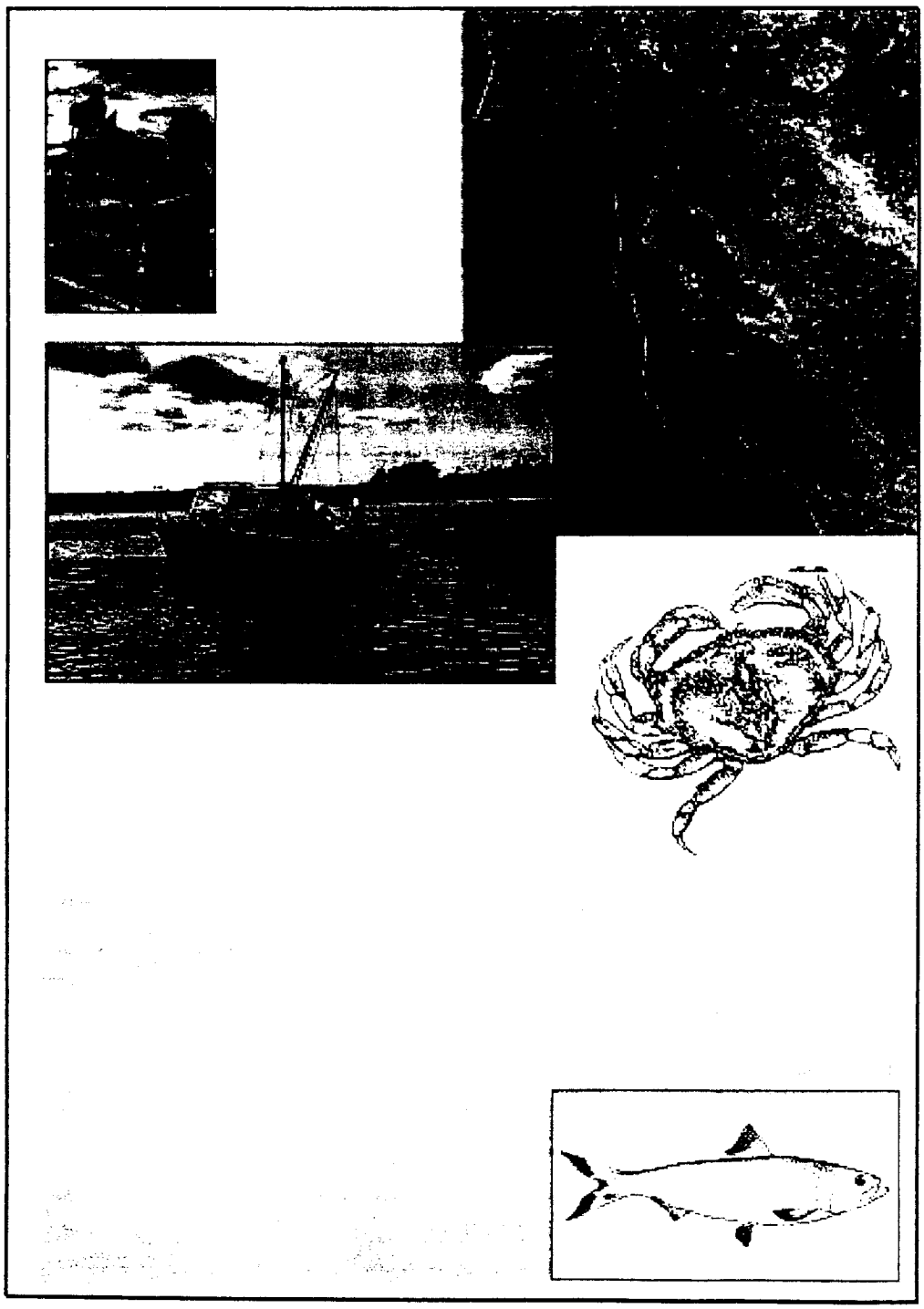
EPA-822-B-00-024
December 2000

EPA

Estuarine and Coastal Marine Waters: Bioassessment and Biocriteria Technical Guidance



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This document is dedicated to the memory of Dr. Douglas Farrell of the Florida Department of Environmental Protection and Dr. Donald Lear, U.S. Environmental Protection Agency (retired). It is fitting that this effort to which they volunteered so much of their invaluable experience and expertise be so dedicated. The benthic community index which Doug developed is also cited here as the "Farrell Index" in further recognition of his unselfish contribution to the protection and management of our coastal resources. Much of the methodology described in the coastal survey portion of this guide was developed from Don Lear's pioneering efforts.

The contributors to this manual sincerely hope that the good common sense, attention to scientific veracity, and practical application of the information to protect our marine resources - so ably personified by Don and Doug - is adequately reflected in these pages.

Disclaimer

This manual provides technical guidance to States, Indian tribes and other authorized jurisdictions to establish water quality criteria and standards under the Clean Water Act (CWA), to protect aquatic life from the effects of pollution. Under the CWA, States and Indian tribes are to establish water quality criteria to protect designated uses. State and Indian tribal decision makers retain the discretion to adopt approaches on a case-by-case basis that differ from this guidance when appropriate and scientifically defensible. While this manual constitutes USEPA's scientific recommendations regarding biological criteria to help protect resource quality and aquatic life, it does not substitute for the CWA or USEPA's regulations; nor is it a regulation itself. Thus, it cannot impose legally binding requirements on USEPA, States, Indian tribes or the regulated community, and might not apply to a particular situation or circumstance. USEPA may change this guidance in the future.

This document has been approved for publication by the Office of Science and Technology, Office of Water, U.S. Environmental Protection Agency. Mention of trade names, products, or services does not convey and should not be interpreted as conveying, official USEPA approval, endorsement or recommendation.

The suggested citation for this document is:

Gibson, G.R., M.L. Bowman, J. Gerritsen, and B.D. Snyder. 2000. Estuarine and Coastal Marine Waters: Bioassessment and Biocriteria Technical Guidance. EPA 822-B-00-024. U.S. Environmental Protection Agency, Office of Water, Washington, DC.

Acknowledgments

The authors wish to express their sincere appreciation to the Estuarine and Coastal Marine Biocriteria Workgroup and the peer reviewers (Arthur Newell - NYDEC, Judith Weis - Rutgers University, John Gentile - University of Miami, Edward Long - NOAA, and Robert Diaz - Virginia Institute of Marine Science).

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Additional scientific, technical, editorial, and production contributions were made by William Swietlik (USEPA), Laura Gabanski (USEPA), Jim Latimer (USEPA), David Scott (Dalhousie University), Gail Chmura (McGill University), Zorana Spasojeviz (McGill University), Jerome Diamond (Tetra Tech, Inc.), Abby Markowitz (Tetra Tech, Inc.), Kristen Pavlik (Tetra Tech, Inc.), Brenda Fowler (Tetra Tech, Inc.), Erik Leppo (Tetra Tech, Inc.), and Regina Scheibner (Tetra Tech, Inc.). This document was prepared by Tetra Tech, Inc.

For the Ocean City/Bethany Beach case study in Section 13.6, thanks are given to David Russell, USEPA Region III provided taxonomic identifications, William Muir, USEPA Region III assisted with data gathering, Eileen Watts, USEPA Region III provided data analysis. Jeroen Gerritsen, Tetra Tech, Inc. offered constructive comments, and Kristen Pavlik, Tetra Tech, Inc. made final editorial changes. Their essential contributions to this report are greatly appreciated.

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Chapter 1
Introduction: Bioassessment and Biocriteria

1.1 Rationale

1.1.1 Water Quality Monitoring

The recognition that chemical water quality analyses do not adequately predict or reflect the condition of all aquatic resources has led to the development of measures of biological integrity expressed by biological criteria. Biological surveys, criteria, and assessments complement physical and chemical assessments of water quality by reflecting the cumulative effects of human activities, and natural disturbances on a water body, including the possible causes of these effects. The biological approach is best used for detecting generalized and non-specific impairments to biological integrity, and for assessing the severity of those impairments. Then, chemical and toxicity tests, and more refined habitat assessments, can be used to identify probable causes and their sources, and to suggest corrective measures.

For the purposes of bioassessment and biocriteria development described here, an estuary is a semi-enclosed water body that has a free connection with the open sea and an inflow of freshwater that mixes with the seawater; including fjords, bays, inlets, lagoons, and tidal rivers. Coastal marine waters are those marine waters adjacent to and receiving estuarine discharges and extending seaward over the continental shelf and/or the edge of the U.S. territorial sea.

1.1.2 Advantages of Bioassessment and Biocriteria

Bioassessment is intended to detect biological responses to pollution and perturbation. Routine water quality monitoring for example, detects effects of nutrient enrichment and chronic acidification, but normally is not designed to detect trace levels of toxicants or contaminants, ephemeral pollution events (e.g., acidic episodes, spills, short-lived toxicants and pesticides, short-term sediment loading), or combined or synergistic impacts. Bioassessment, by monitoring organisms that integrate the effects of environmental changes, may in time detect these effects.

Bioassessment, coupled with habitat assessment; i.e., physical and chemical measurements, helps identify probable causes of impairment not detected by physical and chemical water quality analyses alone, such as nonpoint source pollution and contamination, erosion, or poor land use practices. The detection of water resource impairment, accomplished by comparing biological assessment results to the biological criteria, leads to more definitive chemical testing and investigations which should reveal the cause of the degradation. This, in turn, should prompt regulatory and other management action to alleviate the problem. Continued biological monitoring, with the data collected compared to the criteria, will determine the relative success of the management efforts.

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1.2 Legal Origins

1.2.1 Clean Water Act

The CWA, Section 101, requires federal and state governments to "restore and maintain the chemical, physical, and biological integrity of the nation's waters." Thus, the Act mandates the restoration and maintenance of biological integrity in the Nation's waters. The combination of performing biological assessments and comparing the results with established biological criteria is an efficient approach for evaluating the biological integrity of aquatic ecosystems. Other pertinent sections of the CWA are Sections 305(b), 301(h), and 403(c). Table 1-1 outlines suggestions for the application of biological monitoring and biocriteria for estuaries through existing state programs and regulations.

1.2.2 305(b) Reporting

States and the USEPA report on the status and progress of water pollution control efforts in §305(b) reports submitted every two years. Inclusion of biological assessment results in these reports will improve the public understanding of the biological health and integrity of water bodies. Many of the better known and widely reported recoveries from pollution have involved the renewal or reappearance of valued species to systems from which they had nearly disappeared, or the recovery of a viable fishery from contaminants. Examples of such recoveries are the restoration of the lower Potomac River and of shellfish beds in Maine. Incorporation of biological integrity in §305(b) reports will ensure the inclusion of a bioassessment endpoint, and will make the reports more accessible and meaningful to many segments of the public.

1.2.3 301(h) and 403(c) Programs

Two other programs within USEPA that specifically rely on biological monitoring data in coastal marine areas are the §301(h) Waiver Program and the §403(c) Ocean Discharger Program. The §301(h) program allows estuarine and marine dischargers who meet specific criteria set forth by USEPA to defer secondary treatment if they can show that their discharge does not produce adverse effects on resident biological communities. As part of the modified NPDES permit received through this waiver program, the dischargers are required to conduct extensive biological monitoring programs designed to detect detrimental effects to those biological communities.

The §403(c) Ocean Discharge Program requires that all dischargers to marine waters provide an assessment of discharge impact on the biological community in the area of the discharge and on the surrounding biological communities. This program requires extensive biological monitoring for some dischargers. Community bioassessment methods are valuable in this program for trend assessment and, in some cases, refinement into more rigorous and definitive assessments.

1.2.4 304(a) Criteria Methodology

This technical guidance was developed under the §304(a) requirement that, "criteria for water quality accurately reflecting the latest scientific knowledge of the kind and extent of all identifiable effects on health and welfare including, but not limited to, plankton, fish, shellfish, wildlife, plant life, shorelines, beaches, aesthetics, and recreation which may be expected from the presence of pollutants in any body of

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water . . ." be published and updated as needed.

Under this section, a guidance document must include information on restoration and maintenance of chemical, physical, and biological integrity of navigable and ground waters, waters of the contiguous zone, and the ocean. This also covers information identifying conventional pollutants, such as those classified as biological oxygen demanding,

Table 1-1. Applications of estuarine biological monitoring protocols and biocriteria.

Program	Biological Monitoring and Assessment	Biological Criteria
Section 305(b)/ Reporting	• Improving data for beneficial use assessment. • Improving water quality reporting.	• Identifying waters that are not achieving their aquatic life use support. • Defining an understandable endpoint in terms of "biological health" or "biological integrity" of waterbodies."
National Estuary Program (NEP)	• Assessing status of biological components of estuarine systems. • Develop monitoring objectives and performance criteria. • Establish testable hypothesis and select statistical methods. • Assessing estuarine trophic status and trends, and assessing biological trends. • Select analytical methods & alternative sampling designs. • Evaluate expected monitoring study performance. • Implement monitoring study & data analysis. [Monitoring and sampling needs vary for each estuary]	• Identifying estuaries that are not attaining designated use (including aquatic life use) support. • Defining estuarine biological integrity based on a reference condition. • Identifying impairments due to toxic substances, eutrophication, and habitat modification.
Section 319/Nonpoint Source Program	• Evaluating nonpoint source impacts and sources. • Measuring site-specific ecosystem response to remediation or mitigation activities. • Assessing biological resource trends within watersheds.	• Determining effectiveness of nonpoint source controls.
Watershed Protection Approach	• Assessing biological resource trends within watersheds.	• Setting goals for watershed and regional planning.
TMDLs	• Identifying biological assemblage and habitat impairments that indicate nonattainment of water quality standards. • Priority ranking waterbodies. • Documenting ecological/water quality response as a result of TMDL implementation.	• Identifying water quality-limited waters that require TMDLs. • Establishing endpoints for TMDL development, i.e., measuring success.

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Table 1-1 (cont'd). Applications of estuarine biological monitoring protocols and biocriteria.

Program	Biological Monitoring and Assessment	Biological Criteria
NPDES Permitting	• Measuring improvement or lack of improvement of mitigation efforts. • Developing protocols that demonstrate the relationship of biological metrics to effluent characteristics.	• Performing aquatic life use compliance monitoring. • Helping to verify that NPDES permit limits are resulting in achievement of state water quality standard.
State Monitoring Programs	• Improving water quality reporting. • Documenting improvement or lack of improvement of mitigation efforts including estuary clean-up efforts, TMDL application, NPDES efforts, nonpoint source pollution controls, etc. • Problem identification and trend assessment. • Prioritizing waterbodies.	• Providing a benchmark for measuring effectiveness of controls and performing watershed/regional planning.
Risk Assessment	• Providing data needed to estimate ecological risk to assessment endpoints.	• Providing an assessment or measurement endpoint.
Water Quality Criteria and Standards	• Developing data bases for estuarine phytoplankton, macroinvertebrates, fish, plants, and other assemblages. • Developing indices that assess estuarine biota compared to a reference. • Providing data for aquatic life use classifications.	• Providing benchmark for identifying waterbodies that are not attaining aquatic life use classification. • Developing site-specific standards.
Section 301(h)/ Waiver Program	• Allows marine discharges who meet USEPA criteria to defer secondary treatment if discharge does not produce adverse effects on resident biological communities.	• Providing threshold against which to measure detrimental effects on biological communities.
Section 403(c)/Ocean Discharge Program	• Requires marine dischargers to provide an assessment of discharge impact on biological community in discharge area as well as surrounding communities.	• Providing threshold against which to measure discharger impacts on biological communities.
Section 304(a)/ Criteria Methodology	• Provides information on restoration and maintenance of chemical, physical, and biological integrity of waters. • Identifies conventional pollutants, their concentrations and effects on surrounding communities.	• Providing the benchmark for measuring the effects of pollutants on the biological community.

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suspended solids, fecal coliform, and pH. Section 304(a)(8) authorizes USEPA to develop and publish methods for establishing and measuring water quality criteria for toxic pollution, on other bases than a pollutant by pollutant approach. This includes biological monitoring and assessment methods. Specific states have the authority to enforce more stringent regulations as necessary.

1.2.5 Biocriteria

A major purpose of developing biological assessment methods is to establish biological criteria for surface waters. Biological criteria are guidelines or benchmarks adopted by states to evaluate the relative biological integrity of surface waters. The criteria are defined as "narrative expressions or numerical values that describe the biological integrity of aquatic communities inhabiting waters of a given designated aquatic life use" (USEPA 1990). Biological criteria are, in effect, a practical approach to establishing management goals designed to protect or restore biological integrity. Biocriteria can be adopted by a State into their water quality standards, along with chemical, physical and toxicity criteria to better protect aquatic life uses of waterbodies.

Biocriteria can be developed from reasonable expectations for the locality based on: historical data; reference conditions; empirical models; and the consensus judgment of regional experts (Section 1.4.2). The reference condition component of biocriteria requires minimally impaired reference sites against which the study area may be compared. Minimally impaired sites are not necessarily pristine; they must, however, exhibit minimal influence by human activities relative to the overall region of study (USEPA 1996a). In some instances, "minimally impaired" sites are not available because the entire area has been degraded. Biocriteria are then based on historical data, empirical models if appropriate, and expert judgement to set a condition better than present sites. Restoration of the degraded area must therefore be accomplished before any such reference sites can be established.

Biological criteria typically include the condition of aquatic communities at designated reference sites as an important component. The conditions of aquatic life found at these sites are used to help detect both the causes and levels of risk to biological integrity at other sites of that type in a region. In keeping with the policy of not degrading the resource, the reference conditions—like the criteria they help define—are expected to be upgraded with each improvement to the water resource. It is important that biological criteria not be based on data derived from degraded reference sites. In fact, a concerted effort should be made by States and other jurisdictions to preserve the quality of designated reference sites by setting those areas aside in preserves or parks or by inclusion in use protection programs so that continuity of the biocriteria data base can be maintained. Biocriteria supported by bioassessment surveys serve several purposes in surface water programs, discussed in the following section.

1.3 Uses of Biocriteria

The biocriteria-bioassessment process helps resource managers identify impairment of designated beneficial uses. It expands and improves designated beneficial use classifications and their associated water quality standards. It detects problems other

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survey methods may miss or underestimate. It is a process which helps the resource manager set program priorities. It can also be used to evaluate management and regulatory efforts. For example, the information summarized in Table 1-2 indicates that wastewater outfalls are a controlling factor of soft bottom benthic communities and that there is a moderate scientific understanding of the effects of these outfalls specifically in the Southern California Bight (USEPA 1992).

1.3.1 The Use of Bioassessment Data to Establish Biocriteria Appropriate to Designated Beneficial Uses

The hypothetical information presented in Figure 1-1 represents data collected for a given class of similar estuarine or coastal reaches (e.g., similar sediments, depths, and salinities) within the same geographic region. For these areas some high level of resource quality can be conceived which represents a pristine condition, essentially the optimum potential or integrity of those waters. A completely unimpaired (no negative human impacts upon the organisms of the natural system) estuary or coastal marine area is referred to as having biological integrity. The approximation of this ideal quality at the top of a continuum can be expressed by a variety of environmental measures of the biota indicated on the vertical axis of the graph. The determined **ideal** level of biological measurements at the maximum score is shown by the upper horizontal line (equivalent to biological integrity). A second horizontal line somewhat below this is the level set as the reference condition, the **attainable** level of integrity derived from actual measurements from among the highest quality areas in the class. All information on this axis is expected to be objectively derived through the scientific process and usually is presented in a comprehensive index of many biological characteristics such as an IBI or the EMAP benthic index (Chapter 11).

The horizontal axis represents a progression of socially determined **use designations**; i.e., those predominant uses the State has concluded are appropriate for a particular estuary, region or area within the class. These hypothetical designated uses are arranged on the graph from those usually associated with relatively low water resource quality on the left, to those associated with very high, relatively natural, resource quality on the far right.

The potentially optimal array of **biological criteria** for this class of waters, then, are scores between the reference condition and the level of biological integrity; i.e., between that which is achievable and that which is ideal. The narrower this area, the higher the quality of the waters throughout the class, and the less restoration management is required. The objective, then, is to protect these resources.

On the same horizontal axis, a class of high quality regional uses are further described by a subset of **aquatic life uses**. These are the designated uses for which management goals are also described by desirable characteristics of the aquatic biota to be especially protected, such as "protection of the health and diversity, undiminished, of all indigenous species of fish and invertebrates" for those designated as exceptional natural waters. Resource managers need to apply their first, concerted efforts to those uses because it is usually more cost-effective and resource-conservative to protect existing high quality areas than it is to restore degraded ones.

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Table 1-2. Impacts on the marine environment of the Southern California Bight. Modified from Bernstein et al. 1991.

	Valued Ecosystem Components														
Sources of Perturbation	**Intertidal**	**Phytoplankton**	**Zooplankton**	**Soft Bottom Benthos**	**Hard Bottom Benthos**	**Kelp Beds**	**Wetlands & Estuaries**	**Commercial Shellfish**	**Pelagic Fish**	**Demersal Fish**	**Fish Eggs & Larvae**	**Marine Mammals**	**Marine Birds**	**Human Health**	**All**
Storms	△			▲	●	☆	⊙							○	
El Niños	⬢	★	★	△	○	☆	△		★	▲		○	▲		
California Current		★	★						★	★	☆				
Upwelling		★	★			☆			★		★		●		
Blooms/Invasions				△		○			○	○					Net effect of each source on all components
Ecol. Interactions	★	★	★	●		★	○	▲	⬡		△		⬡		
Power Plants							●		●		○				
Wastewater Outfalls		○		☆	●	○		○		⬢		○	⬡	○	
Dredging				○		○	⬡								
Rivers/Storm Runoff	△					⬡	☆	○		○	●	●	⬡		
Commercial Fishing						△		★	△	⬢		○	○	○	
Sport Fishing								⬢	⬢	⬢		○		⬢	
Habitat Loss/Mod.	☆					⬡	☆	○		○	●	●	⬡		
Oil Spills	☆						▲						⬡	○	
All				Net effect of each source on all components											

KEY	
Potential Importance	**Understanding**
☆ Controlling ⬡ Moderate	■ High
△ Major ○ Some	□ Moderate
	□ Low

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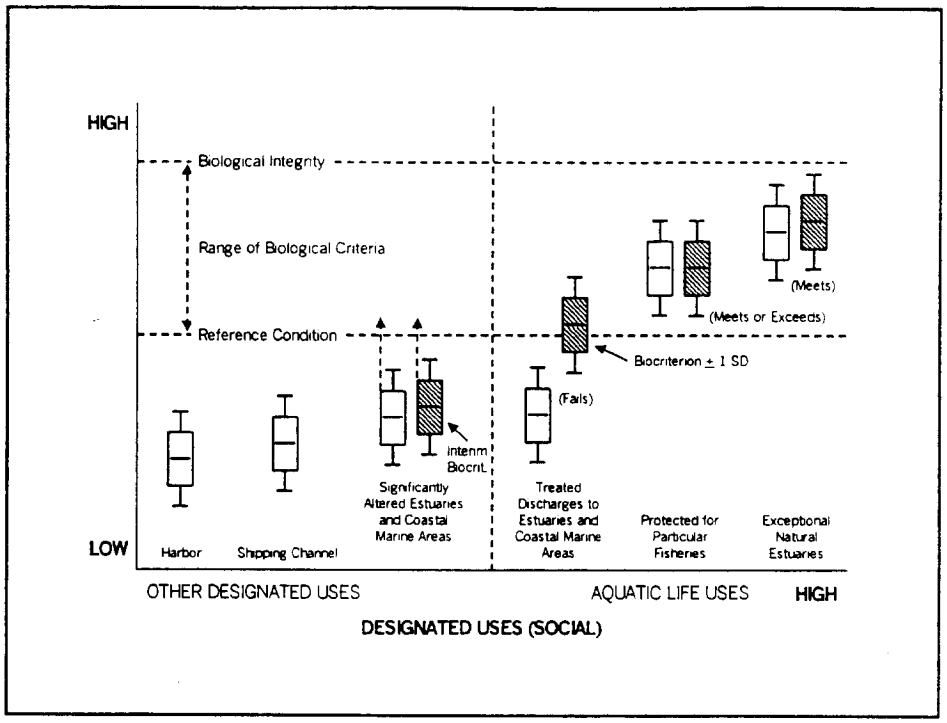
Figure 1-1

Biocriteria for given classifications of estuaries and coastal marine areas. Shaded boxes represent the appropriate biocriterion range for selected classes. Unshaded boxes represent the range of measurement results for test sites in given classes. The vertical arrows above the boxes for the "significantly altered estuaries and coastal marine areas" class indicate the goal of raising the biocriterion for these waters over time in response to restoration efforts.



Selected biocriteria with an acceptable range of variation, perhaps one standard deviation, are shown as cross hatched boxes appropriately located for each designated use. Test results for a given area in any use classification ("box and whisker" plots showin g the full range of measurements including variation for that area) can then be compared graphically to the biocriterion for that designated use. Three interpretations of an estuarine or coastal marine area meets its criterion, meets or perhaps even exceeds its criterion, and fails to meet the criterion are illustrated.

A fourth possible result is the marginal condition of significantly altered systems such as urban harbors or shipping channels. The original condition of these areas may very well have been within the optimal range of biotic health and diversity for the region, but intense development has significantly altered them so that as a group they no longer meet the minimum reference condition for similar areas of the region. An interim biocriterion for these areas may be set with the intention of progressively raising the criterion when sequential restoration efforts are accomplished through a long range management effort.

The "other desi gnated uses" to the left of the bifurcation line may still be surveyed to assist management decision makers; however, they fail to meet the criteria, and there are no designated aquatic life uses which apply.

The designated uses, aquatic life uses and biocriteria are all hypothetical in this illustration, but the interrelationships of societal and scientific elements of decision making should be evident. They are independent processes linked by an environmental ethic and the USEPA policy of antidegradation of water resource quality (the reference condition "bottom line" so to speak). A rational decision can be made which balances that which is ideal with that which is

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achievable measured by the objective processes of science.

1.3.2 Expansion and Improvement of Water Quality Standards

When a State adopts biological criteria in their water quality standards to protect aquatic life uses, the criteria become benchmarks for decision making, and may form the basis for requirements in NPDES permits and other regulatory programs.

1.3.3 Detection of Problems Other Methods May Miss or Underestimate

In the process of establishing biocriteria, more data and information is inevitably developed than was previously available. The review of this new information often reveals problems not evident before or provides expanded insight into existing concerns and issues. Armed with this information, a water resources manager is better able to examine issues and make decisions.

1.3.4 Helping the Water Resource Manager Set Priorities

In light of the new information described above, the schedule of activities, allocation of funds, and uses of personnel and equipment may be more appropriately prioritized according to the urgency or magnitude of the problems identified.

With the expanded available biological information augmenting chemical and physical information, managers can apply a triage approach to water resource projects based on the actual condition of the biota affected. This is much like a physician evaluating multiple emergency medical patients. Essentially, areas that are critically impaired, those that are moderately impaired, and those in good condition for which protection rather than remediation is required, can all be identified. Rational decisions can then be made about how to apply limited resources for the best results in accordance with the needs and priorities of the state.

1.3.5 Use of Biosurveys and Biocriteria to Evaluate the Success or Failure of Management Initiatives or Regulations

The manager may design a biosurvey to collect data before and after a permit, regulation or other management effort has been implemented, perhaps augmented by spatially distributed nearfield/farfield sampling as well. With this information and the biocriteria decision making benchmark, it is possible to clearly evaluate the environmental response of the system to the methods applied. This is useful in the NPDES permit review process as a way to help determine the effectiveness of permit controls. Typically, biocriteria are not used directly in NPDES permits as effluent limitations. Biomonitoring above and below a permit site when compared to the established biocriteria will reveal the adequacy of the permit to achieve its intended purpose.

If the biota are unimpaired or recovering, it may be wise to leave the permit, management practice or regulation as is. If the biota are impaired or declining, the review recommendation may be to change the permit, management technique or regulation accordingly. With NPDES permits, the five year review cycle allows sufficient time for extensive biological information to be developed so this determination can be made with reasonable confidence.

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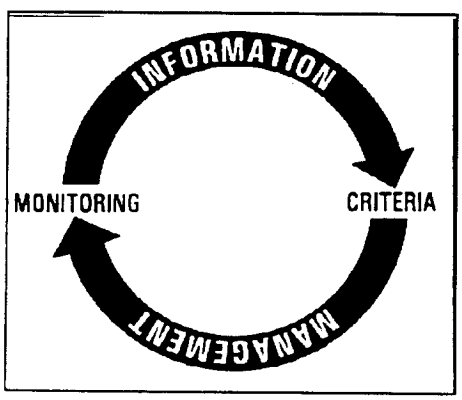
1.4 Program Interdependence

It should be readily evident from the applications described above that physical, chemical, and biological surveys and monitoring (repetitive surveys of the same area) and biological criteria are interrelated in the water resource management process. Figure 1-2 illustrates this interrelationship, often referred to as "adaptive management." In this continually cycling process, monitoring provides the information necessary to identify problems and to establish biocriteria for the decision making, management planning, and implementation necessary to respond appropriately. Continued monitoring then reveals the relative success of the effort by comparing the new results to those criteria again. At this point the criteria or the management plan may be adjusted as needed and the cycle repeats. Ideally, the estuarine or coastal waters improve with each cycle.

Figure 1-2
Program Interdependence



1.5 Implementing Biological Criteria

Implementing biocriteria requires an established and standardized methodology for biological assessment adjusted to regional or state conditions. Hence, guidance for state and regional development of biocriteria has two elements which are described in the biological criteria technical guidance documents such as this one:

- **Bioassessment Protocols** are methods used to assess the status and trends of water bodies. Guidance documents for bioassessment contain suggested methods and protocols for establishing monitoring programs that use biological assessment.

- **Biocriteria Guidance** assists states in establishing biological criteria for water bodies. Biocriteria are a series of ambient water resource quality values or statements of condition that relate to the desired biological integrity for that class of waters. When established they can be used to evaluate similar water bodies in that region. Implementation of biocriteria requires use of bioassessment protocols and a state or regional biomonitoring database. The National Program Guidance for biocriteria describes issues related to development and implementation (USEPA 1990). The first biocriteria technical guidance issued was for streams and small rivers (USEPA 1996a). It incorporated both biosurvey techniques and biocriteria development methods. It was followed by the Lakes and Reservoir Bioassessment and Biocriteria Guidance (USEPA 1998). Each of these documents incorporated biosurvey techniques and the same approach is being followed in similar documents for rivers, wetlands, and coral reefs in addition to this present technical guidance for estuaries and coastal marine waters.

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1.6 Characteristics of Effective Biocriteria

Generally, effective biocriteria share several common characteristics:

- Provide for scientifically sound, cost-effective evaluations;
- Protect sensitive biological values;
- Protect healthy, natural aquatic communities;
- Support and strive for protection of chemical, physical, and biological integrity;
- May include specific characteristics required for attainment of designated use;
- Are clearly written and easily understood;
- Adhere to the philosophy and policy of nondegradation of water resource quality;
- Are defensible in a court of law.

In addition, effective biocriteria are set at levels sensitive to anthropogenic impacts; they are not set so high that sites that have reached their full potential are considered as failing to meet the criterion, nor so low that unacceptably impaired sites are rated as meeting them, which defeats the purpose of the CWA. The establishment of formal biocriteria warrants careful consideration of planning, management, and regulatory goals and the best attainable condition at a site. Balanced biocriteria will allow multiple uses to be considered so that any conflicting uses are evaluated at the outset. The best balance is achieved by developing biocriteria that closely represent the natural biota, protect against further degradation, and stimulate restoration of degraded sites.

Developing and implementing biological criteria occurs in three steps (USEPA 1996a):

1. Planning the biocriteria development program, including:
   - definition of program objectives;
   - establishment of interagency cooperation;
   - identifying acceptable levels of uncertainty for decisions made on the basis of biocriteria;
   - establishing data quality objectives.
2. Characterizing reference conditions for biocriteria and identifying candidate reference sites, which may require a biological survey.
3. Establishing biocriteria based, in part, on characterized reference conditions and designated use classes of the state.

1.7 Conceptual Framework

The central principle of biological assessment is comparison of the biological resources of a water body to a biological criterion based, in part, on a reference condition. Impairment of the water body is judged by its departure from the biocriteria. This approach presumes that the purpose of management is to prevent and repair anthropogenic; i.e., human-induced, damage to natural resources. Biological assessment of water bodies is predicated on our ability to define, measure, and compare biological integrity between similar systems. This requires an

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operational definition of biological integrity as follows:

> "...the condition of the aquatic community inhabiting unimpaired water bodies of a specified habitat as measured by community structure and function (USEPA 1990)."

The functional definition also requires definitions of "unimpaired" and "community structure and function", and the habitat must be specified. Community structure and function is operationally defined by the biological measures chosen for bioassessment, consisting primarily of measures of species richness, trophic diversity (relative numbers of herbivores and top carnivores), and indicator species. In addition to biological community structure and function, chemical (DO, salinity, contaminants, dissolved TOC, inorganic nitrogen, etc.) and physical (sediment composition) attributes are measured to define an unimpaired site. The combined attributes form the basis for defining reference conditions for biological criteria. When unimpaired water bodies do not exist within a region, an operational definition of unimpaired can be developed from a combination of minimally impaired estuaries and coastal waters, historical information, and professional judgment (Section 1.7.2). Figure 1-3 shows a simplified framework for progressing from an estuarine classification to assessing the health of the estuary.

1.7.1 Indicators of Biological Integrity and Survey Protocols

Several analytical approaches have been developed to assess the biological condition of waterbodies within the framework of comparison to reference, ranging in complexity from simple comparison of indicator values, to development of multivariate models:

- Comparison of indicator values – Indicator of metric values can be compared directly to the reference condition, without development of an index. This has been used most often for paleoecological comparison, where biological indicators are limited to certain indicator species, deposition rates, organic carbon loss, etc. (Turner and Rabalais 1994, Sen Gupta et al. 1996, Cooper and Brush 1991, Latimer et al. 1997).

- Multimetric index – The multimetric approach is to define an array of metrics or measures that individually provide limited information on biological status, but when integrated, function as an overall indicator of biological condition. Metrics incorporate information from individual, population, and community levels into a single, ecologically-based index of water resource quality (Gray 1989, Plafkin et al. 1989, Karr 1991). The index is typically a sum or an average of standardized scores of its component metrics (Barbour et al. 1999). Developed initially for streams, the multimetric approach has increasingly been applied to estuaries (Weisberg 1997, Hyland et al. 1998).

- Discriminant analysis to develop an index from metric values – In this approach, metrics (calculated as above) are used to develop a multivariate discriminant analysis model to distinguish reference sites from impaired sites. The calibrated model is then applied to assessment sites to determine whether they are impaired. This approach was used in EMAP-Near Coastal for the

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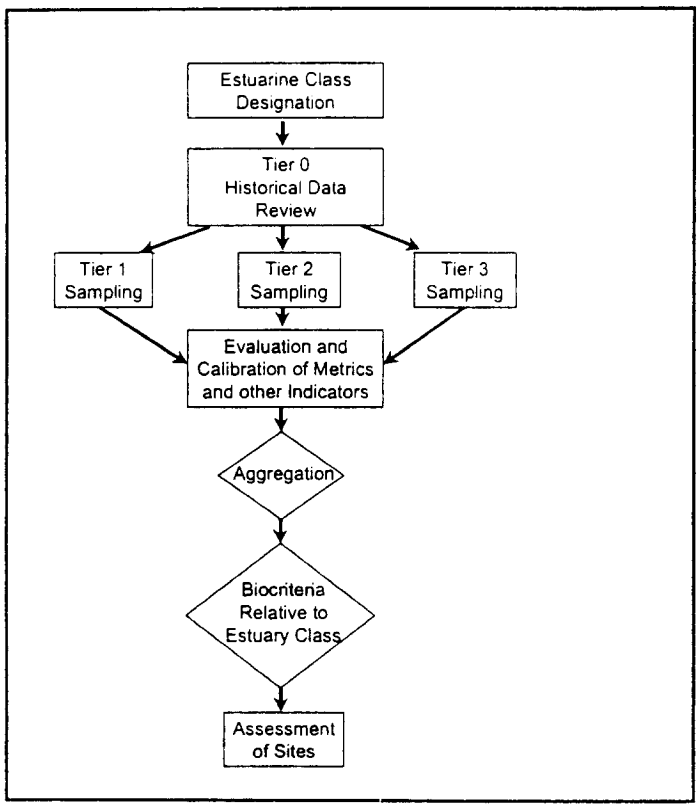

Figure 1-3
The process for progressing from the classification of an estuary to assessing the health of the estuary. Adapted from Paulsen et al. 1991.

Virginian and Gulf provinces (Paul et al. 1999, Engle et al. 1999).

- Multivariate ordination approaches — Several approaches have been developed using multivariate ordination to examine differences in species composition between reference and impaired sites. The purpose of ordination analysis is to reduce the complexity of many variables (for example, abundances of over 100 species from many estuarine sites), by re-ordering the information into fewer variables. These approaches have been used to show the effects of oil drilling in the North Sea (Warwick and Clarke 1991), and to develop an index of benthic quality in California (Smith et al. 2000).

While all of these approaches are appropriate to biocriteria development when properly applied, the multimetric approach is highlighted in this guidance. This is because it is the best developed and most extensively used method to date. Investigators should carefully consider what is most appropriate for their specific program. Time and experience will ultimately determine the best approach or combination for each state to use. Chapter 11 goes into further detail about methods of classification and assessment using all three approaches.

The multimetric concept came to fruition with the fish Index of Biotic Integrity (IBI) first conceived by Karr (1981). The IBI aggregates various elements and surrogate measures of process into a single assessment of biological condition. Karr (1981) and Karr et al. (1986) demonstrated that combinations of these attributes or metrics provide valuable synthetic assessments of the status of water resources.

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A metric is a calculated term or enumeration representing some aspect of biological assemblage structure, function, or other measurable characteristic. Similarly, each of the assemblages (e.g., fish, benthic macroinvertebrates composing the aquatic community) measured would be expected to have a response range to perturbation events or degraded conditions. Thus, biosurveys targeting multiple species and assemblages; i.e., multimetric, will likely provide detection capability over a broad range of impacts, and the biocriteria derived from their results could provide protection to a large segment of the ecosystem.

Metrics can be expressed numerically as integers or ratios. Consistent routines in normalizing individual metric values provide a means of combining metric scores which initially consisted of dissimilar numerical expressions. However, final decisions on impact/no impact or management actions are not made on the single, aggregated value alone. Rather, if comparisons to established reference values indicate an impairment in biological condition, component parameters (or metrics) are examined for their individual effects on the aggregated value and for indications of potential causes.

Assessment of biological integrity using this multimetric approach typically focuses on four broad classes of community properties. Ecological systems respond to anthropogenic impacts with changes in one or more of these classes of properties (e.g., Karr et al. 1986, Schindler 1988, Plafkin et al. 1989, Schindler et al. 1989, Karr 1991, Barbour et al. 1992). The four properties are:

- **Health** of populations, typically expressed as number of individuals per m^2 or as biomass, reflecting possible stress from anthropogenic sources;

- **Community structure and composition**, or the number and kinds of species in an assemblage. Exotic species are typically undesirable, and high diversity is usually desirable. Species structure metrics include diversity and evenness indexes as well as presence of indicator species, counts of tolerant or intolerant species, the percentage of individual taxa in comparison to the total number sampled, and abundance proportions of taxonomic groups (e.g. crustaceans, mollusks, polychaetes), or comparisons of infauna vs. epifauna;

- **Trophic structure**, or the relative proportion of different trophic levels and functional feeding groups (e.g., Barbour et al. 1992). In estuaries, abundant, diverse, and relatively large top carnivores (e.g., piscivorous fish) are typically desirable as representative of a broad, stable, and substantial trophic network;

- **System function**, or the productivity and material cycling of the system or its components (trophic levels, assemblages, species). Measures of system function include primary production and standing stock biomass.

Since biological integrity is defined as an indicator of undisturbed conditions, it too must be measured relative to those conditions. The requirement of the biological criteria process for a reference by which to measure biological integrity makes it a practical tool (*sensu* Peters 1991) for managing society's impact on the natural environment.

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Monitoring and assessment programs typically do not have the resources to measure all ecological attributes of concern to the public and to managers, and assessment tools must be cost-effective. Ideally, metrics selected for monitoring must be scientifically valid; should not require large amounts of expensive equipment; and should be relatively rapid in the field. The selected variables must be:

- **Related to Biological Integrity** In general, almost any biological measurement is related to biological integrity, but some are more clearly tied to the properties of biotic systems of concern to society (e.g., native species, fish production, diverse trophic structure) (Suter 1993);

- **Responsive to Environmental Stresses** Biological measurements and the metrics developed from them must respond to environmental stress. Metrics that are not monotonic; i.e., they do not consistently exhibit low values in response to one end of a stressor continuum and high values in response to the opposite end, or that respond oppositely to different stresses, are difficult to interpret in practice;

- **Measurable with Low Error** Variability and measurement error should be controllable so that a reasonable sampling effort yields sufficient precision. Index period sampling; i.e., sampling during specific time periods in the annual cycle, is one way to reduce seasonal variability. However, there are costs in terms of information derived which may be prohibitive (see later discussion on seasonality);

- **Cost-effective** Cost of a metric should be proportional to the value of the information obtained. Usually, the simplest approach is most cost-effective and should be selected so long as results are sufficient to the agency's objectives;

- **Environmentally Benign to Measure** Sampling methods that significantly disturb or alter habitats and biota should be avoided.

1.7.2 Comparison to a Reference

As noted earlier, establishing biocriteria includes determining the reference condition. The reference condition establishes the basis for making comparisons and for detecting use impairment. Because absolutely pristine estuarine and coastal marine habitats probably do not exist, resource managers must decide on acceptable levels of minimum impacts that exist or that are achievable in a given region. Acceptable reference conditions will differ among geographic regions and states because estuarine salinity gradients, trophic state, bottom sediment types, morphology and biological communities differ between regions.

Reference conditions can be established in a variety of ways. It is important to recognize that the reference condition is best developed from a population of sites, not from a single site. However, in some instances, particularly coastal environments and sites influenced by controversial land uses, the use of site-specific nearfield/farfield stations may be necessary and appropriate to augment the reference condition. They should include information derived from:

- **Historical Data** are usually available that describe biological conditions in

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the estuary or coastal marine region over some period of time in the past. Careful review and evaluation of these data provide insight about the communities that once existed and/or those that may be reestablished. Review of the literature and existing data is an important initial phase in the biocriteria development process. However, if data have not been collected for this specific purpose, they need to be carefully reviewed before being applied;

- **Reference Sites** are minimally impaired locations in the same or similar water bodies and habitat types at which data are collected for comparison with test sites. Reference sites could include sites that are away from point sources or concentrated nonpoint loadings; sites in sub-estuaries; sites occurring along impact gradients (nearfield/farfield); and regional reference sites that may be applied to a variety of test sites in a given area;

- **Models** include mathematical models (logical constructs following from first principles and assumptions), statistical models (built from observed relationships between variables), or a combination of the two. Paleobiological reconstructions of historic or prehistoric conditions are typically statistical or empirical models (Latimer et al. 1997, Alve 1991, Dixit et al. 1992). The degree of complexity of mathematical models to predict reference conditions is potentially unlimited with attendant increased costs and loss of predictive ability as complexity increases (Peters 1991). Mathematical models that predict biological reference conditions should only be used with great caution, because they are complex and often untestable hypotheses (Oreskes et al. 1994, Peters 1991);

- **Expert Opinion/Consensus** A consensus of qualified experts is always needed for assessing all of the above information; establishing the reference condition; and helping develop the biocriteria. This is especially the case in impaired locales where no candidate reference sites are acceptable and models are deemed unreliable. In these cases, expert consensus is a workable alternative used to establish reference "expectations". Under such circumstances, the reference condition may be defined using a consensus of expert opinion based on sound ecological principles applicable to a region of interest. The procedures for these determinations and decisions should be well documented for the record.

1.7.3 Assessment Tiers

Biological surveys of estuaries and coastal marine waters can be implemented in several tiers, ranging from a simple and inexpensive screening to detailed field sampling, analysis, and assessment. The tiered approach gives agencies one suggested approach for planning, organizing, and implementing biological surveys. Other approaches may also be available. Agencies should consider the approach that would work best to meet their program objectives. The tiers are intended to be implemented cumulatively, that is, each tier should incorporate the elements in the preceding tier as appropriate for the waters in which they are applied. Each integrated tier includes both biological and habitat components. Higher tiers require successively more effort and yield more detailed information on

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specific biotic assemblages and potential stresses on the system. Higher tiers reflect higher quality information and reduced uncertainty in the final assessment (Costanza et al. 1992). A desktop screening and three field survey tiers are described in this document. Figure 1-4 provides a summary of the requirements for each tier.

Tier 0 is a desktop screening assessment that consists of compiling documented information for the estuary or coastal marine areas of concern through a literature search and sending survey questionnaires to local experts. No field observations are made at this assessment level. Desktop screening should precede any of the three subsequent tiers. Its purpose is to support the planning for monitoring and more detailed assessments. Information to be compiled in Tier 0 includes: area and geomorphometric classification, habitat type, watershed land use, population density, NPDES discharges, water quality data (salinity, temperature, DO, pH, turbidity), biological assemblage data, and water column and bottom characteristics.

Tier 1 is the least complex of the survey approaches. It consists of a one-time visit to sites during a suitable, predetermined index period to collect biological and habitat data using standardized methods. The focus of this tier is on developing screening or survey information. These variables include a rudimentary identification of organisms (benthos, fish, macrophytes, or phytoplankon), water column characteristics (salinity, temperature, DO, pH, Secchi depth, water depth), and bottom characteristics (grain size, RPD layer depth, total volatile solids, and sediment toxicity). States may choose some variation of this list depending on regional characteristics and resources. Evaluation of the data collected, as well as historical data for the area, leads to an initial classification of sites and identification of candidate reference sites.

Tier 2 is somewhat more complex. A higher level of detail is incorporated into the standardized biological methods and multiple visits to the site are made to address temporal variability and/or seasonality. Another assemblage (epibenthos) could be selected in addition to those listed above. Water column nutrient measurements are added to the Tier 1 water column characteristics. A tactile categorization of grain size, plus total organic carbon, are added to the bottom characteristics. The data collected in this tier will allow the development of preliminary biological criteria.

Tier 3 is the most rigorous survey tier. Three or more assemblages are sampled here, through multiple site visits to account for seasonal variations in the selected estuarine and coastal marine biological assemblages and should incorporate supplemental studies which might be necessary for diagnostic assessment of the potential causes of observed impairments. This tier adds water column pesticides and metals measurements, plus full grain size characterization (sieving to determine percent grain size composition), acid volatile sulfides, and sediment contaminants. This tier also allows the resource agency to develop a database sufficient to support resource management activities to reduce the identified impairments and to develop and refine biocriteria.

Biological Assessment

The procedure of biological assessment is to sample two or more biological assemblages and record data such as abundance, condition, biomass, and

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Figure 1-4
General comparison of Tiered Approach. Tiers are intended to be implemented cumulatively. Each tier should incorporate the elements in the preceding tier as appropriate for the waters in which they are applied, as necessary for specific programs.

Tier 0
- -No field observations
- -Desktop screening
- -Literature search
- -Questionnaires to local experts
- -Support planning for monitoring and more detailed assessments

Tier 1
- -One time visit to sites during suitable, predetermined index period
- -Least complex survey approach
- -Develop screening/survey information
- -States choose variation of variables (assemblages + water column & bottom characteristics) according to regional characteristics & resources
- -Leads to initial classification & ID of candidate reference sites

Tier 2
- -2 or more visits to site
- -More complex
- -Possible to add another assemblage
- -Add to water column & bottom characteristics samples
- -Allows for development of preliminary biological criteria

Tier 3
- -4 or more visits to sites
- -Most rigorous
- -3 or more assemblages
- -Incorporate supplemental studies
- -Additions to water column & bottom characteristics
- -Develop database to support resource management activities to reduce impairments & define/refine biocriteria

other characteristics of each species. These data are then used to calculate metrics, such as taxa richness, percent dominance, number of intolerant species, and percent abundance of tolerant species. Each metric is compared to its expected value under reference conditions, and rated good (similar to reference), fair (different from reference), or poor (substantially different from reference). Numeric scores are assigned to the ratings, and the scores of all metrics of an assemblage are summed for a total score for the assemblage. The total score is again compared to the expected total score under reference conditions, and the assemblage as a whole is assigned an ordinal rating of good, fair, or poor. This second comparison to reference conditions is necessary because not all metrics are expected to score "good" at all times even in pristine conditions; the final assemblage score thus takes into account natural variability in metric values.

Once these values are satisfactorily established they can be incorporated in the development of a biocriterion for a particular estuarine or coastal marine

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class. "Biological assessment" at this point becomes a comparison of monitoring scores to the biocriteria for management decision making. The following several chapters describe the processes necessary to the development of suitable metrics and finally their incorporation in biological criteria for water resource management decision making.

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Chapter 5

Sampling Program Issues Biological Assemblages and Design

This chapter presents sampling program issues that are common to each of the three assessment tiers that employ field sampling. These issues include the biological assemblages (Section 5.1) that might be sampled, sampling design strategies (Section 5.2 and 5.3), and logistical considerations (Section 5.4). Historically, benthic macroinvertebrates have been the most widely sampled assemblage, which is described in detail in this chapter.

As described earlier in this document, a possible sampling methodology is a progressive tiered design, ranging from simple biological assessment to detailed, intensive studies. The tiers are intended to be implemented cumulatively, that is when possible, each tier should incorporate the elements in the preceding tier as appropriate for the estuaries or coastal marine water in which they are applied. In general, the methods are derived from those used along the coastal United States (Dauer 1993, Farrell 1993a, b, Nelson et al. 1993, Word 1980, 1978, Word et al. 1976); in Puget Sound (Eaton and Dinnel 1993); in the EMAP - Estuaries program (Holland 1990), and in USEPA's National Estuary Program (NEP) (USEPA 1992) and 403 Monitoring Program (USEPA 1994a).

Assessment tiers 1 through 3 require sampling biological assemblages and habitats in one or more field visits. Six biological assemblages, including two developmental/experimental assemblages, are recommended for estuarine and coastal marine waters bioassessment. Each tier is comprised of a subset of assemblages, with the number of assemblages increasing in the higher tiers. While these six assemblages are described, specific environmental circumstances and budget constraints will determine what subset each state uses. For example, if finances are extremely limited on the East Coast the single most effective assemblage to sample may be macrophytes. On the West Coast benthic macroinvertebrates or fish may the assemblages of choice. The bioassessment measurements are made along transects extending from shore to the deepest (channel) portion of the estuary, in a systematic grid along transects extending away from point source discharges (nearfield/farfield), or in a probablistic design. The number of transects or grid points, the assemblages sampled, and the intensity of sampling effort are determined by the assessment tier with overall effort increasing at each higher tier.

5.1 Assemblages

The study of any group of organisms will yield information on the status of their environment. The objectives in selecting assemblages for estuarine and coastal marine bioassessment were to identify those that: (1) are unambiguously useful for biological assessment; (2) can be sampled and interpreted in a cost-effective way; and (3) have easily calculated metrics that can be used alone or in a multimetric index of the assemblage. Assemblages

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that meet these criteria are suggested for use in estuarine and coastal marine assessment; assemblages that do not presently meet the criteria are considered to be developmental. Suggested assemblages include infaunal benthic macroinvertebrates, fish, aquatic macrophytes, and phytoplankton (chlorophyll *a*). The developmental assemblages include zooplankton, epibenthos, and paleoenvironmental systems. These developmental assemblages are promising, but they lack the same level of refinement documented for the suggested assemblages listed above and unresolved technical problems remain with respect to cost-effective assessment and interpretation. Background and rationale for these suggested assemblages was presented in Chapter 2.

Multimetric bioassessment is not a ready-made, one-size-fits-all instrument that will tell managers whether estuaries or coastal marine waters are healthy. It is an approach that is expected to be modified to specific regional conditions before it can be applied. For example, bioassessment of streams has been successful when modified and calibrated regionally (e.g., Barbour et al. 1996a, Ohio EPA 1990, Miller et al. 1988), but it has been less successful when used "off-the-shelf." Successful application requires region-specific selection and calibration of metrics, as well as regional characterization of reference conditions. For example, benthic infauna are rare in rocky, fjord-type estuaries and would be an inappropriate assemblage to sample in such a setting.

5.1.1 Benthic Macroinvertebrates (Infauna)

Benthic macroinvertebrates are an appropriate assemblage for all biological assessments of water bodies because they respond to water, sediment, and habitat qualities (Holland 1990, Plafkin et al. 1989), are not very mobile, and consequently, integrate long-term changes in these ecosystem components. For those reasons, benthic macroinvertebrates tend to dominate this text.

Individual macroinvertebrate species have sensitive life stages that respond to stress and integrate effects of short-term environmental variations, whereas community composition depends on long-term environmental conditions. In addition to taxonomic identification, benthic macroinvertebrate metrics may require knowledge of the feeding group to which a species belongs, for example, suspension feeders and deposit feeders. Potential metrics for estuarine and coastal marine benthos are listed in Table 5-1. Metrics considered in the EMAP Estuaries program are listed in Table 5-2.

Sampling Strategies

The sampling area should focus on the most predominant substrate available (in many estuaries and coastal marine areas this will be soft sediments of mud through sand grain sizes), and the metrics should be developed independent of microhabitat variation (Table 5-3). The type of sampling gear will depend on the substrate being sampled; each substrate has its own optimal sampling gear (Section 5.1.1.4). Standardized sampling techniques for each gear type should be followed to allow for the comparison of data. Processing of samples should be standardized by using a mesh size appropriate to the region. In the past, monitoring programs conducted in east coast waters have often used a 0.5-mm mesh screen, while west coast programs have used a 1.0-mm screen (Bowman et al. 1993). States should consider testing various mesh size screens to determine

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Table 5-1. Potential benthic macroinvertebrate metrics.

Metric	Response to Impairment
No. of taxa	reduced
Mean no. of individuals per taxon	substantially lower or higher
% contribution of dominant taxon	elevated
Shannon-Wiener diversity	reduced
Total biomass	substantially lower or higher
% biomass of opportunistic species	elevated
% abundance of opportunistic species	elevated
Equilibrium species biomass	reduced
Equilibrium species abundance	reduced
% taxa below 5-cm	reduced
% biomass below 5-cm	reduced
% carnivores and omnivores	elevated
No. of amphipod species	reduced
% individuals as amphipods	reduced
% individuals as polychaetes/oligochaetes	elevated
No. of bivalve species	reduced
% individuals as molluscs	reduced
% individuals as deposit feeders	elevated
Mean size of organism in habitat	reduced
Proportion of expected no. of species in sample	reduced
Proportion of expected no. of species at site	reduced
Mean weight per individual polychaete	reduced
No. of suspension feeders	reduced
% individuals as suspension feeders	reduced
No. of gastropod species	reduced
No. of Capitellid polychaete species	elevated

the most appropriate size for their bioassessment activities. Ferraro et al. (1994) present a process to evaluate the optimum infaunal sampling protocol; i.e., sampling unit area, sieve mesh size, and sample size [*n*], discussed more fully in Section 5.2.6.

Time and Costs

An informal survey of some states that conduct routine monitoring of estuaries and coastal marine waters indicates that estuarine sampling requires a minimum of two full-time equivalent (FTE) staff,

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Table 5-2. Metrics from which the EMAP Virginian and Louisianian benthic indexes were developed. Louisianian Province has reduced number of metrics due to knowledge gained from previous Virginian province studies (n.a. - not applicable).

Community Measure of Structure/ Function	Metrics
Virginian Province	
Biodiversity/ Species Richness	Proportion of expected number of species present in a sample ■ Proportion of expected number of species present at a site ■ Shannon-Weiner Diversity Index ■ Pielou's evenness index
Abundance Measures	Total benthic abundance per event ■ Mean benthic abundance per sample ■ Total benthic biomass per event ■ Mean benthic biomass per sample
Individual Health	Biomass/abundance ratio ■ Mean weight per individual polychaete ■ Mean weight per individual mollusc
Functional Groups	Number of suspension feeding organisms per event ■ Biomass of suspension feeding organisms per event ■ Percent of total benthic abundance as suspension feeders ■ Percent of total benthic abundance as suspension feeding biomass ■ Number of deposit feeding organisms per event ■ Biomass of deposit feeding organisms per event ■ Percent of total benthic abundance as deposit feeding organisms ■ Number of benthic omnivores/predators per event ■ Biomass of benthic omnivores/predators per event ■ Percent of total benthic abundance as omnivores/predators ■ Percent of total benthic biomass as omnivores/predators ■ Number of opportunistic species per event ■ Mean number of opportunistic species per sample ■ Percent of total benthic abundance as opportunists ■ Number of equilibrium species per event ■ Mean number of equilibrium species per sample ■ Percent of total benthic abundance as equilibrium species ■ Percent of mean benthic abundance as equilibrium species
Taxonomic Composition	Number of amphipods per event ■ Amphipod biomass per event ■ Percent of total benthic abundance as amphipods ■ Percent of total benthic biomass as amphipods ■ Number of bivalves per event ■ Bivalve biomass per event ■ Percent of total benthic abundance as bivalves ■ Percent of total benthic biomass as bivalves ■ Number of gastropods per event ■ Gastropod biomass per event ■ Percent of total benthic abundance as gastropods ■ Percent of total benthic biomass as gastropods ■ Number of molluscs per event ■ Mollusc biomass per event ■ Percent of total benthic abundance as molluscs ■ Percent of total benthic biomass as molluscs ■ Number of polychaetes per event ■ Polychaete biomass per event ■ Percent of total benthic abundance as polychaetes ■ Percent of total benthic biomass as polychaetes ■ Number of Capitellid polychaetes per event ■ Percent of total benthic abundance as Capitellid polychaetes ■ Number of Spionid polychaetes per event ■ Percent of total benthic abundance as Spionid polychaetes ■ Percent of total polychaete abundance as Spionid polychaetes ■ Number of Tubificid oligochaetes per event ■ Percent of total benthic abundance as Tubificid oligochaetes
Louisianian Province	
Biodiversity/ Species Richness	Shannon-Wiener Diversity Index ■ Pielou's Evenness Index ■ Mean number of species ■ Mean number of polychaete species
Abundance Measures	Mean benthic abundance per site
Individual Health	n.a.
Taxonomic Composition	Mean abundance of amphipods per site ■ Proportion of total benthic abundance as amphipods ■ Mean abundance of decapods per site ■ Proportion of total benthic abundance as decapods ■ Mean abundance of bivalves per site ■ Proportion of total benthic abundance as bivalves ■ Mean abundance of gastropods per site ■ Proportion of total benthic abundance as gastropods ■ Mean abundance of molluscs per site ■ Proportion of total benthic abundance as molluscs ■ Mean abundance of polychaetes per site ■ Proportion of total benthic abundance as polychaetes ■ Mean abundance of Capitellid polychaetes per site ■ Proportion of total benthic abundance as Capitellid polychaetes ■ Mean abundance of Spionid polychaetes per site ■ Proportion of total benthic abundance as Spionid polychaetes ■ Proportion of total polychaete abundance as Spionid polychaetes ■ Mean abundance of Tubificid oligochaetes per site ■ Proportion of total benthic abundance as Tubificid oligochaetes

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Table 5-3. Sampling summary for infaunal benthic macroinvertebrates.

Habitat	Preferred: soft sediments (mud-sand).
Sampling Gear	Regionally most appropriate for substrate (Table 5-4).
Index Period	Regionally most appropriate Preferred: Summer - East & Gulf Coast Spring - Pacific Northwest Alternative: All four seasons, or winter and summer
Sampling	Preferred: samples from 3 grabs at each of at least 10 sites. Alternative: keep sites as replicates if a within-class variance estimate will be used in assessment.
Analysis	Preferred: lowest practical taxonomic level Alternative: identification to class and family.

and has an associated per sample cost of $200 - $400.

Coastal marine sampling requires a minimum of four FTEs, and has an associated cost of $400 - $800. Three months to a year are required from time of sampling to preparation of an interpretive report.

Assessment Tiers

The benthic infaunal assemblage is appropriate for all three field tiers outlined for the biological assessment of estuaries and coastal marine waters. Tier 1 determines the presence/absence of macroinvertebrates below 5-cm depth in the sediment and briefly describes the class and family of observed benthos. Tier 2 determines the major taxa and indicator species present in each sample to the genus and species level. Tier 3 applies a full benthic community assessment, recording the numbers of individuals in each grab to the genus and species level, and can include determination of biomass if deemed appropriate by the state. Tier 3 uses the benthic community assessment with replication and additional diagnostic stations and parameters as indicated by the data.

Gear Type

All sampling methods and gear types have specific biases because they capture a target assemblage. Because estuaries and coastal marine waters are complex environments with a potentially large number of habitats, it is important to choose sampling methods and gear appropriate for a specific habitat type. Sampling within a given habitat type such as a salinity regime, bottom grain size, and/or depth should be conducted so that samples can be considered representative of the community being studied.

A large number of benthic sampling methods and gear are available. The choice of appropriate methods and gear will depend upon the goals of the sampling and the habitat to be sampled.

- In subtidal areas, benthic infauna can be collected using grabs, such as Young, Ponar, or Van Veen; or cores such as box, gravity, or hand-held cores collected by divers. Grab or

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core size and number of replicates should be sufficient to adequately sample the infaunal community, bearing in mind that distribution is usually spatially clumped rather than random or regular; and

- Intertidal areas may best be sampled at low tide with hand-held cores. For certain infauna it may also be feasible to estimate abundance by counting the number of surface structures within a given area. For example, some polychaete worms build identifiable tube or mound structures, or leave identifiable fecal coils in intertidal areas. If the local infauna has been studied to the extent that identification of such topographic features can be correlated to the presence of a particular organism, crude abundance and presence/absence evaluations may be possible.

Collection of sediments and benthic organisms should be done concurrently in order to reduce the costs of field sampling and to permit sound correlation and multivariate analyses. Therefore, the sampling equipment and procedure should also include sampling the sediment.

Desirable attributes for sediment sampling gear include:

- Creates a minimal pressure wave when descending;
- Forms a nearly leakproof seal when the sediment sample is taken;
- Prevents winnowing and excessive sample disturbance when ascending;
- Allows easy access to the sample surface so that undisturbed subsamples may be taken;
- Allows vertical sectioning of undisturbed samples for profile examination.

Penetration well below the desired sampling depth is preferred to prevent sample disturbance as the device closes. It is best to use a sampler that has a means of weight adjustment so that penetration depths may be modified with changing sediment type (USEPA 1992).

Grab Samplers

Well designed and constructed grab samplers are capable of consistently sampling bottom habitats. Depending on the size of the device, areas of 0.02- to 0.5-m^2 and depths ranging from 5- to 15-cm may be sampled. Limitations of grab samplers include:

- Variability among samples in penetration depth depending on sediment properties;
- Oblique angles of penetration which result in varying penetration depths within a sample; and
- The sample may be folded or otherwise distributed by some devices, such as the Shipek sampler, resulting in the loss of information concerning the vertical structure of benthic communities in the sediments.

However, careful use of these devices will provide reliable quantitative data. Grab samplers are the tools of choice for a number of estuarine and marine monitoring programs due to their ability to provide quantitative data at a relatively low cost (Fredette et al. 1989, USEPA 1986-1991). Various grab samplers which could be used for Tiers 1-3 are summarized in Table 5-4.

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Table 5-4. Summary of bottom sampling equipment (Adapted from USEPA 1992, Klemm et al. 1992, and ASTM 1998b).

DEVICE	USE	ADVANTAGES	DISADVANTAGES
KB Corer	Soft sediments only.	Samples a variety of soft substrates up to harder types. Sampling tube can be modified up to 100-cm^2 substrate surface; least disturbance to water/bottom interface. Can be used in shallow to medium-shallow water up to 30.5-m or deeper.	Samples limited surface area. Requires boat and winch.
Ballcheck Single and Multiple Tube	Soft sediment only.	Good penetration on soft sediment. Small sample volume allows greater number of replicates to be collected in a short time period. Samples deep burrowing organisms. Used in shallow to deep water (3-m to 183-m). Automatic check valves prevent sample loss.	Heavy; requires boat and winch. Does not retain sand unless bronze core retainers are used.
Fluorocarbon plastic or Glass Tube	Shallow wadeable waters or deep waters if SCUBA available. Soft or semi-consolidated deposits	Preserves layering and permits historical study of sediment deposition. Rapid-samples immediately ready for laboratory shipment. Minimal risk of contamination.	Small sample size requires repetitive sampling. Impractical in water > 1-m depth if SCUBA not available.
Hand Corer with removable Fluorocarbon plastic or glass liners.	Same as above except more consolidated sediments can be obtained.	Handles provide for greater ease of substrate penetration. Above advantages.	Careful handling necessary to prevent spillage. Requires removal of liners before repetitive sampling. Slight risk of metal contamination from barrel and core cutter.
Box Corer	Same as above.	Collection of large undisturbed sample allowing for subsampling.	Hard to handle.
Phleger (Gravity) Corer	Semi-consolidated sediments.	Low risk of sample contamination. Maintains sediment integrity relatively well.	Careful handling necessary to avoid sediment spillage. Small sample, requires repetitive operation and removal of liners. Time consuming.
Young Grab	Lakes, estuarine and marine areas.	Eliminates metal contamination if grab is plastic or kynar lined. Reduced pressure wave. Can subsample. Better penetration in sand than the modified Van Veen.	Expensive, heavy, requires boat and winch.
Ekman or Box Dredge	Soft to semi-soft sediments. Can be used from boat, bridge, or pier in waters of various depths. Weights can be added for deeper penetration in fine sand.	Obtains a larger sample than coring tubes. Can be subsampled through box lid. Hinged top doors reduce washout, shock waves and substrate disturbance. Range of sizes available.	Possible incomplete jaw closure and sample loss. Possible shock wave which may disturb the fines. Metal construction may introduce contaminants. Possible loss of fines on retrieval. Inefficient in deep water or where even moderate current exists.

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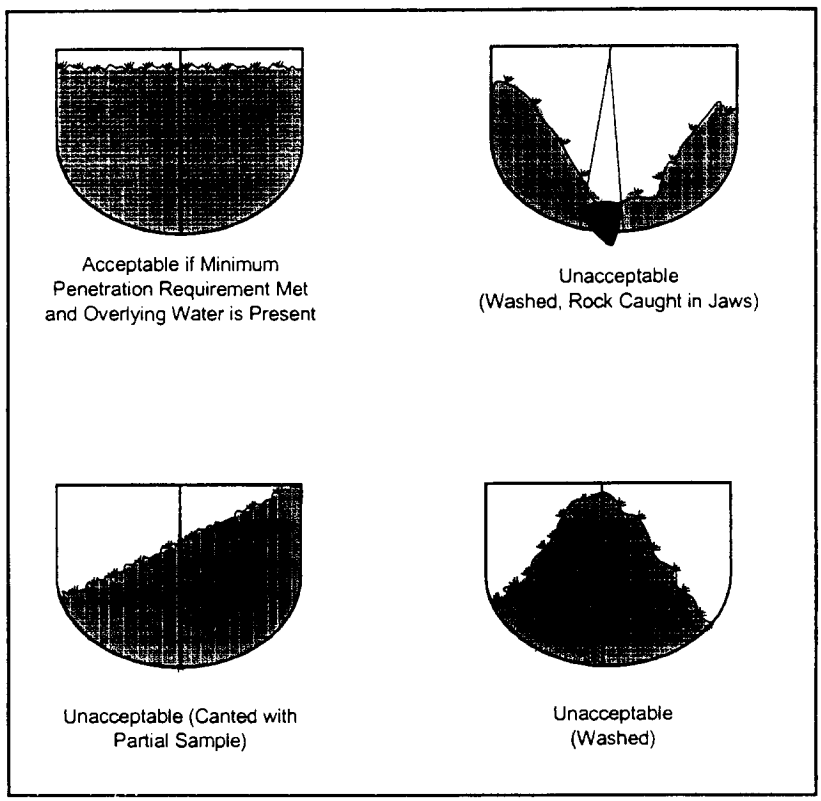
Table 5-4 (Cont'd). Summary of bottom sampling equipment (Adapted from USEPA 1992, Klemm et al. 1992, and ASTM 1998b).

DEVICE	USE	ADVANTAGES	DISADVANTAGES
Ponar Grab Sampler	Useful on sand, silt, or clay.	Most universal grab sampler. Adequate on most substrates; very efficient for hard sediments. Large sample obtained intact permitting subsampling. Better penetration than other grabs; sideplates and screens reduce washout, shock waves and substrate disturbance.	Shock wave from descent may disturb fines. Possible incomplete closure of jaws results in sample loss. Possible contamination from metal frame construction. Sample must be further prepared for analysis. A very heavy grab requires use of a boat with winch and cable. Shell hash can hold jaws open causing loss of sample. Must use stainless-lined grab for sediment metals samples.
BMH-53 Piston Cover	Waters of 1-2-m deep when used with extension rod. Soft to semi-consolidated deposits.	Piston provides for greater sample retention.	Cores must be extruded on site to other containers - metal barrels introduce risk of metal contamination.
Modified Van Veen	Useful on sand, silt, or clay.	Adequate on most substrates. Large sample obtained intact.	Requires boat and winch. Shock wave from descent may disturb fines. Possible incomplete closure of jaws results in sample loss. Possible contamination from metal frame construction. Sample must be further prepared for analysis. Limited penetration in hard sand. Possible overpenetration in soft silt.
BMH-60	Sampling moving waters from a fixed platform.	Streamlined configuration allows sampling where other devices could not achieve proper orientation.	Possible contamination from metal construction. Subsampling difficult. Not effective for sampling fine sediments.
Smith-McIntyre Grab	Useful on most substrates.	Reduced pressure wave. Designed for sampling hard substrates. Can subsample and make vertical cross-sections. Greater penetration in sand and cobble than modified Van Veen, but possibly not as deep as a Young grab. Better closure in areas with wood debris.	Loss of fines. Heavy; requires boat and winch. Possible metal contamination unless grab is lined.
Scoops, Drag Buckets	Useful on most substrates. Various environments depending on depth and substrate.	Inexpensive, easy to handle.	Loss of fines on retrieval through water column. Layer information not collected.

The number and kinds of macroinvertebrates collected by a particular grab may be affected by the habitat sampled, substrate type sampled, depth of penetration, angle of closure, completeness of closure of the jaws and potential loss of sample material during retrieval, creation of a "shock" wave and "washout" of organisms at the surface of the substrate. The high-flow velocities often encountered in rivers and wave action in estuaries and coastal marine waters can also affect stability of the sampler (Klemm et al. 1992). USEPA EMAP-Estuaries protocols describe a simple and consistent method for accepting or rejecting a bottom grab (Figure 5-1).

The type and size of the grab samples (or other device) selected for use will depend on factors such as the size of boat, available winch and hoisting gear, the type of sediment to be sampled, water depth, current velocity, and whether sampling is conducted in sheltered areas or open water (Klemm et al. 1992). The EMAP-Near Coastal

Figure 5-1
Cross-section of sediment in clamshell bucket illustrating acceptable and unacceptable grabs.



Program selected a Young grab (sometimes referred to as a Young-modified Van Veen) that samples a surface area of 440-cm^2 (Weisberg et al. 1993). This Young grab was selected because it deploys easily from small boats (24-ft) and it samples sand and mud habitats adequately. The maximum penetration depth of the grab was 10-cm.

PONAR Grab:

The PONAR has side plates and a screen on the top of the sample compartment to prevent loss of the sample during closure. With one set of weights, this heavy steel sampler can weigh 20-kg. Word et al. (1976) report that the large amount of surface disturbance associated with Ponar grabs can be greatly reduced by simply installing hinges rather than fixed screen tops, which will reduce the pressure wave associated with the sampler's descent into the sediment. The standard Ponar takes a sample area of 523-cm^2. A small version, the petite Ponar grab, takes a sample area of 232-cm^2 and can be used in habitats where there may be an unusual abundance of macroinvertebrates, thus eliminating the need to subsample.

The weight of the standard Ponar grab makes it necessary to use a winch and cable or portable crane for retrieving the sample, and ideally the samples should be taken from a stationary boat. The smaller version (petite Ponar grab) is designed for hand-line operation, but it may be used with a winch and cable.

Ekman Grab:

The Ekman grab sampler is used to obtain samples of macroinvertebrates from soft sediments, such as very fine sand, mud, silt, and sludge where there is little current. This grab is inefficient

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in deep waters, under adverse weather conditions, and in waters with moderate to strong currents or wave action. The Wildco box corer is like a heavy duty Ekman with a frame and weights and can be used to collect macroinvertebrates in estuaries. Because of its weight a winch is necessary for retrieving the sample from a stationary boat.

The Ekman grab sampler is a box-shaped device with two scoop-like jaws that must penetrate the intended substrate without disturbing the water-sediment boundary layer, close when positioned properly on the bottom, and retain a discrete sample of sediment while it is brought to the surface for processing. Hinged doors on the top of the grab prevent washout during sample retrieval. The grab is made of 12- to 20-gauge brass or stainless steel and weighs approximately 32-kg. The box-like part holding the sample has spring-operated jaws on the bottom that must be manually set. The sampler is available in several sizes; however, in very soft substrates only a tall model should be used, either a 23-cm or a 30.5-cm model. The Ekman grab can be operated from a boat with a winch and cable.

Smith-McIntyre Grab:

The Smith-McIntyre grab sampler is designed to obtain samples of macroinvertebrates from sediments in rough weather and deep water in estuaries and oceans. This device samples a surface area of 0.1-m^2 and is useful for sampling macroinvertebrates from a broad array of sand, gravel, mud, clay, and similar substrates.

The Smith-McIntyre grab sampler has hinged top doors to prevent sample washout and the pressure wave in descent. Its paired jaws are forced into the intended substrate by two "loaded" strong coiled springs when the grab touches the bottom. The jaws close when positioned properly on the bottom, and retain a discrete sample of sediment to be brought to the surface for processing. The device is heavy and can weigh 45.4-kg or more. The chief advantage of the sampler is its stability and easier control in deep and rough waters. The spring-loaded jaws of the Smith-McIntyre grab must be considered a hazard and caution should be exercised when using the device. Due to the weight and size, this device must be used from a vessel with boom and lifting capabilities.

Modified Van Veen Grab:

The modified Van Veen grab sampler is used to obtain samples of macroinvertebrates from sediments in estuaries and other marine habitats. This device is useful for sampling sand, gravel, mud, clay and similar substrates and is available in three sizes: 0.06-m^2, 0.1-m^2, and 0.2-m^2. Larger versions of this grab are available, and their use is dependent upon the type of bottom to be sampled, and the type of vessel available to deploy the sampler.

The modified Van Veen grab sampler has paired jaws that penetrate the intended substrate without disturbing the water-sediment boundary layer. They are closed by the pincher-like action of two long arms. The long arms give added leverage for penetrating hard sediments.

The modified Van Veen is basically an improved version of the Petersen grab in that long arms have been attached to the jaws to help stabilize the grab on the bottom in the open sea just prior to or during closure of the device. This grab is used extensively in Puget Sound for the ambient monitoring program and for pollution-related surveys. Large hinged screen doors with rubber flaps have

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been added to the top of the sampler for access to the surface of the sample. Additional weights can be applied to the modified Van Veen jaws to effect greater penetration in sediments, although penetration is not as deep in hard sand or cobble as with the Young grab or the Smith-McIntyre.

Young Grab:

The Young grab sampler is similar in operation to the Van Veen and the Smith-McIntyre, but the sample can be accessed undisturbed from the top of the grab through hinged doors like a Smith-McIntyre. It is encircled by a ring-like frame which enhances flat, stable landings of the grab on the substrate. Weights can be added to the frame to aid penetration in hard sand or cobble. A major advantage of the Young grab is efficient performance without the risk of injury associated with the spring-loaded Smith-McIntyre. This grab can be provided in a 0.044-m^2 and a 0.1-m^2 version. The former is appropriate to small boat operations while the latter size is more effective for marine work and obviously requires fewer lowerings or "drops" to obtain the same volume of material and community representation.

Recent comparisons of the Young and Smith-McIntyre grabs in rough Atlantic waters revealed consistently greater volumes of sediment collected by the Young grab in six trials each in soft sandy muds, sand, packed sand, and sand and gravel sediments. While the grabs were the same size (0.1-m^2) and had the same weight attached, the significant factor in performance was the design differences of the two grabs (Gibson 1995, unpublished).

While either the 0.1-m^2 Young or Smith-McIntyre designs are effective off-shore grabs for the biocriteria development purposes of this guidance, the Smith-McIntyre provides better access to the sample while the Young grab is easier and safer to operate, especially in rough weather. An advantage of both designs is that the retrieved sample can be cross-sectioned and examined intact, although this is easier with the Smith-McIntyre design.

Core Samplers

Core samplers use a surrounding frame to ensure vertical entry; vertical sectioning of the sample is possible (USEPA 1986-1991). Coring devices can be used at various depths in any substrate that is sufficiently compacted so that an undisturbed sample is retained; however, they are best suited for sampling the relatively homogenous soft sediments, such as clay, silt, or sand of the deeper portions of estuaries and coastal marine waters. Because of the small area sampled, data from coring devices are likely to provide very imprecise estimates of the standing crop of macrobenthos.

KB, Ballcheck, and Phleger Corers:

KB type, Ballcheck, and Phleger corers are examples of devices used in shallow or deep water; they depend on gravity to drive them into the sediment. The cores are designed so that they retain the sample as it is withdrawn from the sediment and returned to the surface. Hand corers designed for manual operation are used in shallow water. Sections of the core can be extruded and preserved separately or the entire core can be retained in the tube and processed in the field or laboratory. Intact cores can also be preserved by freezing and processed later.

Additional replication with corers is feasible because of the small amount of material per sample that must be handled in the laboratory. Multiple-

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head corers have been used in an attempt to reduce the field sampling effort that must be expended to collect large series of core samples (Flannagan 1970).

The Dendy inverting sampler (Welch 1948) is a highly efficient coring-type device used for sampling at depths to 2- or 3-m in nonvegetated substrates ranging from soft mud through coarse sand. Because of the small surface area sampled, data obtained by this sampler suffer from the same lack of precision (Kajak 1963) as the coring devices described above. Since the per-sample processing time is reduced, as with the corers, large numbers of replicates can be collected.

Stovepipe-type devices include the Wilding sampler (Wilding 1940, APHA 1992) and any tubular material such as 60- to 75-cm sections of standard 17-cm diameter stovepipe (Kajak 1963) or 75-cm sections of 30-cm diameter aluminum irrigation pipe fitted with handles. In use, the irrigation pipe or commercial stovepipe is manually forced into the substrate, after which the contained vegetation and coarse substrate materials are removed by hand. The remaining materials are repeatedly stirred into suspension, removed with a long-handled dipper and poured through a wooden-framed floating sieve. Because of the laborious and repetitive process of stirring, dipping, and sieving large volumes of material, the collection of a sample often requires 20- to 30-minutes.

The use of stovepipe samplers is limited to standing or slowly moving waters having a maximum depth of less than 60-cm. Since problems relating to depth of sediment penetration, changes in cross-sectional area with depth of penetration, and escape of organisms are circumvented by stovepipe samplers, they are appropriate for quantitative sampling in all shallow-water benthic habitats and can be deployed from small boats. They probably represent the only quantitative device suitable for sampling shallow-water habitats containing stands of rooted vascular plants and they will collect organisms inhabiting the vegetative substrates as well as those living in sediments.

In marine waters, benthic macrofauna are generally collected using various box cores deployed from ships or other platforms, or diver operated cores. A box coring device consisting of a rectangular corer having a cutting arm which can seal the sample prior to retraction from the bottom should be used. In order to sample a sufficient number of individuals and species, and to integrate the patchy distribution of fauna, each sample should have a surface area of no less than 100-cm^2 and a sediment depth of at least 20-cm. In sediments having deep, burrowing fauna, a box corer capable of sampling deeper sediment may be needed. In sandier sediments, it may be necessary to substitute a grab sampler for the box corer in order to achieve adequate sediment penetration. Visual inspection of each sample is necessary to insure that an undisturbed and adequate amount of sample is collected.

Sieve Mesh Size

The use of different sieve mesh sizes for screening benthic samples limits the comparability of results between marine monitoring studies (Reish 1959; Rees 1984). The major advantage of using a smaller mesh size is the retention of both juvenile and adult organisms as well as large-and small-bodied taxa. The major disadvantage is the concomitant increased cost of sample processing. For example, using a 0.5-mm mesh rather than a 1.0-mm mesh could increase

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Table 5-5. Mesh sizes used in estuary benthic monitoring programs.

Monitoring Program	Mesh Size (mm)	Reference
Chesapeake Bay	0.5	Dauer 1993, Holland et al. 1987, 1988, 1989, Ranasinghe et al. 1992
Tar/Pamlico	0.5	Eaton 1992a-d
EMAP-Near Coastal	0.5	USEPA 1992, Weisberg et al. 1993, Holland 1990
Naples Bay, Florida	0.595	Simpson et al. 1979
Puget Sound Ambient Monitoring Program	1.0	PSWQA 1988, 1990, 1991
Puget Sound Estuary Program	1.0	Simenstad et al. 1991

contracting these identifications is justified based on the information obtained and the assessment tier to which it would be applied. One approach to this problem of obtaining sufficient taxonomic expertise is for the states of a region to cooperate in a joint venture to employ the taxonomic expertise necessary to all. In this manner the cost of a skilled taxonomist, either contracted or on staff, can be shared.

5.1.2 Fish

Fish communities include species in a variety of trophic levels (omnivores, herbivores, planktivores, piscivores). Fish are long-lived, integrate long- and short-term changes, and they also integrate effects of lower trophic levels; thus, fish community structure is a good measure of integrated environmental health. Estuarine and coastal marine fish receive a large amount of public attention because of sport and commercial fishing and attendant concerns regarding fish production and safety for human consumption. On the negative side, fish may be wide-ranging or migratory and might not reflect local conditions in estuaries and coastal marine waters; some fish species may also be influenced by management (stocking), angling, and commercial harvesting; and unbiased sampling is difficult because each feasible gear type is highly selective.

Sampling Gear

Fish communities may vary considerably among the numerous habitat types that may be present in a target estuary or coastal marine area. The choice of sampling method and gear type will depend upon the habitat and the fish species of interest. Shallow areas may best be sampled using dip nets or beach seines, while deeper waters may be sampled using gill nets, purse seines, or otter trawls. Net and mesh size should be appropriate to allow a representative sample of target fish to be obtained. Fishing effort should be comparable among stations with constant tow distances, times, speeds, and lengths of trawl warps. Because there is no easy way of estimating population size in any given area of an estuary or coastal marine area, consistency in effort is of the utmost

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retention of total macrofaunal organisms by 130 to 180%; however, costs for processing the samples may increase as much as 200% (USEPA 1986-1991).

It is recommended that a standard mesh size be selected for all monitoring studies. A review of estuarine monitoring programs from around the country (Bowman et al. 1993) showed that both 0.5- and 1.0-mm mesh sizes are used, with a slight majority of the programs reviewed using a 0.5-mm mesh screen (Table 5-5). Dauer (1993) evaluated biocriteria developed from data collected as part of the Virginia Benthic Biological Monitoring Program using a 0.5-mm mesh screen.

Sieving can be done either aboard the survey vessel or on shore after the cruise. Sieving occurs prior to fixation (sample preservation) aboard the vessel, whereas waiting until after the cruise requires fixation prior to sieving. If inadequate concentrations of fixative are added and deterioration or decomposition of organisms occurs, there may be significant sample degradation. If large numbers of samples are to be collected, field sieving reduces sample storage requirements as well as the modification/loss of data (USEPA 1992, 1994d).

After samples have been collected, the samples must be processed so that data can be collected and analyzed. Two aspects of sample processing of particular concern are the subsampling and identification that may occur in the field or laboratory. Sorting procedures are described in Klemm et al. (1992).

- Subsampling of benthic infauna can be accomplished by subcoring; i.e., removing smaller core samples from within a grab or core sample, and sorting all organisms found within the subcore. The size and number of subcores that should be taken will depend upon the variability of the infaunal community. Representative subsampling can be difficult to achieve if benthic species have patchy or clumped distributions. Subsampling can also damage collected organisms (e.g., polychaete worms), decreasing the number of specimens that can be identified to genus or species;

- Several studies have examined the effect of varying levels of taxonomic analysis on the results of statistical measures of the infaunal community (e.g., Ferraro and Cole 1990, 1992, 1995, Warwick 1988, Warwick et al. 1990). The studies indicate that in some instances species-level taxonomic identification does not yield any more information than family- or even phylum-level identification. The degree of taxonomic proficiency required to adequately characterize the community will depend upon the diversity present in the community. Species level identification is necessary and cost-effective for fish surveys. However, while this is desired for macroinvertebrates, it is often too costly and assessment needs can usually be met at the genus level.

Although species-level identifications may not be necessary for classifying sites as minimally impaired or impaired, this degree of taxonomic identification may be required to assess the sources of impairment using data collected in Tier 3. Species-level identifications require greater taxonomic expertise than do higher taxonomic divisions; this species level of expertise may not be as readily available to state agencies. If this is the case, then state resource managers must determine whether the cost of

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importance to allow legitimate comparisons among sites.

Maryland DNR's IBI sampling techniques are designed to sample the nearshore fish communities in the tidal tributaries of the Chesapeake Bay. They were modeled after the Maryland Striped Bass Juvenile Seine Survey which has been ongoing since 1954 (Goodyear 1985). Two beach seines are pulled at each site allowing a half hour interval between hauls for repopulation of the seine area. Seines are pulled with the tide employing a "quarter sweep" method where one end of the seine is held on shore while the other end is fully extended perpendicular to shore, and then pulled back into shore forming a semi-circle. The seine used is a bagless 6.4-mm mesh seine 30.5-m in length and 1.2-m deep. Precautions are taken upon approaching the site to avoid disturbance of the sampling area.

Concurrent trawls are pulled with the tide in the channel adjacent to shore. A small otter trawl (3.1-m with 12.8-mm stretch mesh, and 50.8-cm x 25.4-cm doors) with tickler chains is used to sample the bottom community local to the seine sample area. Water quality measurements (temperature, dissolved oxygen, pH, conductivity, and salinity) and Secchi depth are also taken in the trawling area. Water quality is sampled at surface, mid, and bottom depths. These measurements have proven useful in relating water quality parameters to fish communities. A summary of fish sampling is given in Table 5-6.

- Subsampling of fish collected using any of the sampling methods mentioned above is problematic. It is probably most efficient and statistically valid to identify and make external measurements and observations of all fish caught during a given tow or time period.

- The level of taxonomic identification required to effectively characterize the fish community will depend upon the diversity of the community being sampled and the metrics being used to evaluate the data. Identification to species is preferable for most individuals taken in a given area. Individuals that cannot be field-identified should be preserved and returned to the lab for identification.

5.1.3 Aquatic Macrophytes

Macrophytes form an integral part of the littoral zone of many estuaries and

Table 5-6. Sampling summary for fish.

Habitat	Sublittoral.
Sampling Gear	Seines and any gear that effectively captures bottom-feeding and pelagic fish, usually otter trawls.
Index Period	Any season can be selected depending upon migration and recruitment patterns in the region. Seasonal sampling might be needed to assess particular problems.
Sampling	Bottom-feeding and pelagic fish. Sufficient sets of gear to obtain representative species counts (usually 4 or more).
Analysis	Collected species are weighed, measured, and examined for external abnormalities (lesions, growths, deformities). Histopathology may be performed.

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coastal marine waters, serving as habitat for fish and invertebrates as well as being a distinct biological assemblage. For many estuaries, the areal extent and distribution of SAV is used as an indicator of estuarine quality (Batiuk et al. 1992). Ecosystems whose primary producer component is dominated by aquatic macrophytes can be transformed to macro algae or phytoplankton-dominated systems through nutrient enrichment. Increased nutrient input stimulates macrophyte growth; however, it also promotes growth of periphyton and phytoplankton, which shade the SAV. The shading reduces macrophyte growth and survival (Dennison et al. 1993, Batiuk et al. 1992). Overall, macrophyte standing stock is an excellent indicator of estuarine water quality. The presence of confounding factors, such as diseases, can be determined from examination of affected plants, or from historical information. Potential macrophyte metrics are listed in Table 5-7 and the recommended sampling protocol for macrophytes is summarized in Table 5-8. Field sampling can be performed in a single visit. Plants are identified and weighed on-site, with voucher specimens preserved as necessary. There is no intensive laboratory analysis required.

5.1.4 Phytoplankton

Phytoplankton are the base of most estuarine food webs (Day et al. 1989), and fish production is linked to phytoplankton primary production (e.g., Day et al. 1989). Excessive nutrient and organic inputs from human activities in estuaries and their watersheds leads to eutrophication characterized by: reduction in seagrasses, increases in phytoplankton biomass, macrophyte biomass (macroalgal biomass), reduced water clarity, and reduced oxygen saturation in bottom waters. From a human perspective, problems might include loss of aesthetic appeal, decreases in desirable commercial and game fishes, and loss of recreational access caused by increased macrophyte production.

Phytoplankton standing stock is measured by surface chlorophyll *a* concentration, sampled at the 0.5-m depth at each sampling site (Table 5-9). Tiers 1 and 2 can use a single measurement taken at each sampling site with a fluorometer attached to a conductivity-temperature-depth meter (CTD) (USEPA 1994c) taken from June through September. Alternatively, chlorophyll *a* may be determined spectrophotometrically on phytoplankton samples returned to the lab. Tier 2 can include identification of dominant taxa, including nuisance taxa. Tier 3 uses a seasonal or annual average surface chlorophyll concentration from all stations over all sampling events and can include full characterization of the phytoplankton community.

If phytoplankton communities are to be sampled, several techniques may be employed; these are described more fully in APHA (1992).

- Phytoplankton samples may be obtained using water bottles deployed on a wire at a given, or preferably various, depths. The water bottles used should be constructed and cleaned in a manner appropriate for the collection of phytoplankton samples (e.g., Niskin bottles washed and rinsed in order to remove contaminants). Chlorophyll concentration is measured from the sampled water, and phtyoplankton cells may be filtered or settled for identification and enumeration.

Table 5-7. Potential aquatic macrophyte metrics.

Metric	Response to impairment
Tier 1:	
% cover	substantially more or less than reference
dominant taxa	substantially more or less than reference
Tiers 2-3:	
% cover	reduced or enhanced
biomass	substantially more or less than reference
maximum depth of plant growth	reduced under enrichment
density of new shoots	reduced
stem counts	reduced

Table 5-8. Sampling summary for aquatic macrophytes.

Habitat	Euphotic zone.
Sampling Gear	Aerial photography; quadrats
Index Period	During growing season
Sampling	Tier 1: Estimate of area covered by macrophytes. Tiers 2-3: Quadrat samples for biomass collected by diver; 3-5 randomly placed transects perpendicular to shore; samples are taken at 0.5-m depth intervals from edge of emergent zone to the sublittoral.
Analysis	Tier 1: Dominant taxa identified, % cover estimated from aerial photography. Tiers 2-3: All species identified, relative abundance of each estimated from wet weight.

Table 5-9. Sampling summary for phytoplankton.

Habitat	Each sampling site preferred.
Sampling Gear	Fluorometer attached to CTD (USEPA 1994e) for *in situ* measurements; or spectrophotometrically on water samples collected with a water sampler.
Index Period	Tiers 1 and 2: June - September Tiers 2 (optional) and 3: growing season average; 6-10 samples; March - October (longer in subtropical regions).
Sampling	Preferred: single sample, 0.5-m depth. Alternate: at same depths as nutrient samples.
Analysis	Tier 1: Chlorophyll *a* mg/L (Tiers 1-3). Tier 2: ID dominant taxa. Tier 3: full community species characterization.

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- Phytoplankton may also be collected by net hauls using a plankton net with an appropriate mesh size.

Bottle collections are most useful when analyzing a bulk community measure such as chlorophyll *a* concentration (assuming a fluorometer coupled to a CTD is not used), while net hauls are better for studies designed to enumerate species. Water samples for chlorophyll *a* determination can also be used for nutrient analysis.

- The level of taxonomic identification that should occur will depend upon the diversity of the community, the analyses that are to be performed, and the cost and availability of taxonomic experience;

- If phytoplankton are collected using water bottles, the water may be subsampled in the field or lab prior to analysis. The size and number of subsamples that should be taken will depend upon the variability present in the community;

- If subsamples are taken from net hauls, it may be necessary to resuspend the organisms found in the cod end of the net in a larger volume of water in order to facilitate subsampling.

5.1.5 Zooplankton (Developmental)

Zooplankton are most effectively sampled using net hauls with 118-µm mesh sizes. Because zooplankton are known to exhibit diel periodicity in their locations in the water column, sampling times should reflect this temporal variability; i.e., sampling should, in general, be conducted at night. Also, consideration should be given to the use of vertical or oblique tows. In any instance, gear size, mesh size, rate of retrieval on the haul back, vertical or oblique tow, time of day or night and tide cycle are factors which must be kept constant if zooplankton surveys are to be included in biocriteria development.

Meaningful bulk community measurements do not exist for zooplankton; therefore, if zooplankton are to be sampled, they should be identified and enumerated. It may be difficult to locate or develop the taxonomic expertise necessary to identify zooplankton to species, especially given the large number of planktonic larvae. Zooplankton are considered to be in a developmental status with respect to their use as an estuarine and coastal marine bioassessment assemblage. Zooplankton populations experience year-round seasonal fluctuations in abundance as a result of variable larval recruitment into the population, variable food sources, and physical processes which may move larvae and adults into and out of the estuary (Day et al. 1989). The pattern of seasonal abundance differs with changes in latitude. Zooplankton in higher latitudes have one or more mid-summer peaks and very low numbers during the winter.

Abundances in temperate estuaries are much more variable and may experience spring peaks and minima during the summer and winter months. Tropical estuaries do not experience the low in population during the winter.

Some long-term monitoring projects have identified community measures that indicate changes in

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environmental conditions over time (e.g., nutrient loads or toxicants), as well as particular zooplankton taxa whose densities affect larval fish survival (Buchanan 1991). Zooplankton community characteristics that are under investigation for application as bioindicators include:

- Diversity, measured through standard indexes such as Shannon-Wiener, to evaluate the taxonomic complexity of the assemblage;
- Ratios of specific taxonomic groups within the assemblage to gauge community balance and identify possible impairment;
- Presence of Hypotrichs (a ciliate of the order Hypotrichida);
- Total biomass to assess assemblage production;
- Relative abundance of pollution tolerant and sensitive species to identify and evaluate impairments to the assemblage;
- Unnatural variability in abundance can be used to identify the presence of short-term pollution or climate events;
- Size structure can be used to evaluate the growth of cohorts in the assemblage, which can provide information on possible short- and long-term system perturbations.

5.1.6 Epibenthos (Developmental)

The epibenthos assemblage is also considered to be in a developmental stage for use in estuarine and coastal marine bioassessment. Taxa within the epibenthic community appear to be persistent and sensitive to environmental stress. They are characterized by physiological mechanisms that allow them to tolerate the varying salinity, DO, and temperature conditions encountered in estuaries and coastal marine waters, or reproductive cycles that allow them to avoid high-stress periods. Some epibenthos and facultative infauna can relocate to avoid areas of environmental stress.

Epibenthos can be sampled using a Renfro beam trawl, otter trawl, or epidbenthic sled. Camera tows or remotely operated vehicles with camera or video capabilities may also allow enumeration of epibenthos, although collection of organisms would not be possible and quantitative assessments difficult. Subsampling might involve a process similar to that suggested by Plafkin et al. (1989); a box with a numbered grid system into which collected epibenthos are evenly distributed could be used to randomly select an appropriate number of organisms for subsequent sorting.

Some of the advantages to using epibenthos for estuarine and coastal marine bioassessment are:

- This assemblage is very sensitive to anthropogenic sources of stress, and it can be used in both a nearfield and farfield context with equal facility;

Sampling can be conducted in shallow waters using a dip net and in deep waters with a trawl;

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- The total number of common species will be limited by the fact that the deep water sampling gear is restricted to fairly level bottoms;
- Subsampling can be employed to reduce labor costs and increase cost-effectiveness;
- Field and lab work, and data analyses can be done quickly with trained personnel;
- Samples can be sorted qualitatively, and a nonparametric analysis can be applied to provide a quick screening method.
- Seagrasses and macroalgae can hinder or increase the time necessary for field sorting;
- The seasonality of epifauna needs to be factored into the sampling design.

The developmental method described in Chapter 13 appears promising for detecting impairment. If successfully adapted to regions outside Florida, North Carolina, and Puget Sound where it is being tested, it may become a standard estuarine bioassessment method in the future. A proposed sampling protocol is summarized in Table 5-10.

Table 5-10. Sampling summary for epibenthos.

Habitat	Soft sediments (sleds and trawls); shallow, vegetated (dip net)
Sampling Gear	Renfro Beam Trawl (Farrell 1993a,b), small otter trawl; epibenthic sled; dip net
Index Period	Preferred: mid-summer Alternative: growing season, average of 10 samples.
Sampling	Ca. 4-m tow length in estuaries; 0.1 - 0.5 nm tow lengths (DGPS) in coastal waters and Puget Sound.
Analysis	Taxonomic ID preferably to species.

The disadvantages of this assessment methodology are:

- The stress index is developed solely for anoxia; it might not allow assessment of other stressors;
- Stress values may not be available for many species, or may be difficult to determine;
- Sleds and trawls are restricted to level bottoms; and cannot be used for sampling hard bottoms, or rock rubble;

5.1.7 Paleoenvironmental Systems (developmental)

Diatom and foraminifera species have narrow optima and tolerances for many environmental variables, which make them useful in quantifying environmental characteristics to a high degree of certainty. They immigrate and replicate rapidly, which makes them quick to respond to environmental change (Dixit et al. 1992). Changes in assemblages also correspond closely to shifts in other biotic communities sampled in estuaries such as aquatic macrophytes,

zooplankton, and fish. They have also been used alone as environmental indicators of eutrophication, metal contamination, salinification, thermal effluents, and land use changes. Furthermore, since diatoms and foraminifera are abundant in almost every marine ecosystem, a relatively small sample is sufficient for analysis. This allows for many samples to be easily collected, analyzed, and archived (Dixit et al. 1992).

The general lack of time-series data has prompted attempts to demonstrate marine eutrophication from present-day observations using the benthic community and chemical criteria (Dale et al. 1999). Benthic foraminifera have been proven useful as indicators of oxygen concentration in bottom sediments (Alve 1991). Dinoflagellate cysts are also increasingly useful as indicators of short-term environmental change caused by climate and human pollution (Dale et al. 1999). The cysts are recovered by pollen identification techniques; they are acid-resistant and therefore not subject to dissolution problems sometimes affecting diatoms and foraminifera (Dale et al. 1999). Measurements of biogenic silica in sediments are most often used as an index of diatom production (Stoermer et al. 1990, Conley et al. 1993, Cooper 1995). Isolation of BSi from Si in mineral phase is based upon the fact that the silica of diatoms is only weakly crystalline and dissolves readily in a weak base. Potential indicators and a proposed sampling summary are shown in Tables 5-11 and 5-12.

The total number of cores taken in a particular estuary is dependent upon the hydrological complexity of the estuary. Generally, one to three cores, but some times up to ten are required from each estuary or tributary being assessed. However, once a paleoecological record is established, there is no need to repeat the sampling.

Although the number of cores is small, each core requires substantial effort to analyze: sectioning, radioisotope dating, chemical analysis, pollen analysis for further dating, and diatom or foraminifera analysis. Current estimates for paleological analysis is about $100 per section (not per core), depending on the number and intensity of analysis done on each section and the experience of the lab performing the analysis. The complexity of estuaries requires some background information about the area in which sampling is occurring. This information should assist in decision making on the location and number of cores to be retrieved.

The study of paleoenvironmental systems requires a corer that will retrieve an intact core, with minimal edge disturbance (Table 5-4). K-B, Phleger, and Piston corers have all been used successfully for these analyses (see Section 5.1.1). Small surface area is not an issue; a single core will suffice.

5.2 Sampling Design Issues

Consideration of sampling design is critical in developing a new monitoring program for estuarine bioassessment and biocriteria. Sampling design includes defining the questions to be addressed by the data, defining the units that will be

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Table 5-11. Potential paleoecological indicators

Indicator	Response to Impairment	Reference
Taxa richness (diatom, foraminifera, dinoflagellate cysts)	reduced	Cooper and Brush 1991
Biogenic silica	increase with nutrient enrichment	Turner and Rabalais 1994
Total organic carbon, Total N, Total S	increase with enrichment	Turner and Rabalais 1994
Ammonia/Elphidium ratio (foraminifera)	increase with hypoxia	Sen Gupta et al. 1996
Centric/pennate ratio (diatoms)	increase with nutrient enrichment	Cooper and Brush 1991
% *Cyclotella*	increase with nutrient enrichment	Cooper and Brush 1991
sedimentation rate	increase with watershed erosion	Brush 1989
Dinoflagellate cysts	increase with cultural eutrophication	Dale et al. 1999
% *Fursenkoina*	increases with hypoxia	Alve 1991
% *Trochammina*	increases with hypoxia	Patterson 1990

Table 5-12. Sampling summary for paleoenvironmental systems

Habitat	Stable depositional zone, biogeochemical conditions for preservation
Sampling gear	Bottom corer
Index period	None
Sampling	Tiers 1-2: none
	Tier 3: background information specific to the estuary being sampled will determine the number of cores necessary.
Analysis	Cores sectioned at regular intervals depending on deposition rate and resolution desired.
Diatoms Foraminifera Dinoflagellate Cysts	Species composition and enumeration of at least 300 organisms in each section. Digestion/clarification methods depend on assemblage.
Age of sections up to 150 years	^{210}Pb determination based on radioisotope assay with alpha spectroscopy.
Older than 150 years	Palynological (pollen) analysis correlated with known historical changes in terrestrial vegetation (land use), and ^{14}C analysis (>1000 yr).

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sampled, and developing a sampling design that is cost-effective for answering the defined questions.

5.2.1 Statement of the Problem

The first task in developing a sampling and assessment program is to determine, and be able to state in simple fashion, the principal questions that the sampling program will answer. Questions may or may not be framed as hypotheses to test, depending on program objectives. For example, suppose that a sampling program objective is to establish reference conditions for biological criteria for estuaries in a given region. Typically, the initial objectives of a survey designed to develop criteria are to identify and characterize classes of reference sites in estuaries. Initial questions may then include:

- Should minimally disturbed sites be divided into two or more classes that differ in biological characteristics and dynamics?;
- What are the physical, chemical, and relevant biotic characteristics of each of the estuary site classes?

After the monitoring and assessment program has developed biological criteria, new questions need to be developed that encompass assessments of individual sites, estuaries, or estuaries of an entire region or state. Specific questions may include:

- Is site abc similar to reference sites of its class (unimpaired), or is it different from reference sites (is it altered or impaired)?;
- Overall, what is the status of estuarine waters in the region? What percentage of estuarine waters is similar to reference conditions? What percentage is impaired?;
- Has estuary abc changed over a certain period? Has it improved or deteriorated?;
- Overall, have estuarine waters in the region improved or deteriorated over a certain period? Have individual estuaries improved? Are more waters similar to reference conditions now than some time ago?

Finally, resource managers often wish to determine the relationships among variables, that is, to develop predictive, empirical (statistical) models that can be used to design management responses to perceived problems. Examples of specific questions include:

- Can trophic state of an estuary be predicted by areal nitrogen loading rate?;
- Can biota of an estuary be predicted by watershed land use?

Monitoring and assessment data, and derived models, may also be used to help determine causal relationships between stressors and responses of systems. Inferring cause requires manipulative experiments, or inference from multiple lines of evidence (Suter 1993). Since surveys and monitoring programs preclude experimental investigations, inference of causal relations is beyond the scope of this document. Often, there is enough experimental evidence available from other studies so that additional causal experiments are not necessary and would be superfluous (e.g., current knowledge of nutrients and trophic state generally makes it

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unnecessary to "prove" experimentally which nutrients are limiting). The development of predictive models usually does not require formal hypothesis testing.

It is also necessary to specify the units for which results will be reported. Usually, these units are the population (e.g., all estuarine waters), but often subpopulations (e.g., embayments or tributaries of a given estuary) and even individual locations (e.g., sites of special interest) can be used. In order to help develop the sampling plan, it is useful to create hypothetical statements of results in the way that they will be reported, for example:

- Status of a place: *Baltimore harbor is degraded;*
- Status of a region: *20% of the area of Puget Sound has elevated trophic state, above reference expectations;* or *20% of estuaries in Oregon have elevated trophic state;*
- Trends at a place: *Benthic species richness in Baltimore harbor has increased by 20% since 1980;*
- Trends of a region: *Average estuary trophic state in New Jersey has increased by 20% since 1980;* or *Average benthic index values in 20% of estuaries of the west coast have increased by 15% or more since 1980;*
- Relationships among variables: *50% increase of N loading above natural background is associated with decline in taxa richness of benthic macroinvertebrates, below reference expectations;* or *Estuaries receiving runoff from large urban areas have 50% greater probability of elevated trophic state above reference than estuaries not receiving such runoff.*

5.2.2 Definition of the Assessment Unit

Defining the resource and assessment unit of the resource begins the process of developing biological criteria. An "assessment unit" is a whole estuary or part of an estuary, that will be assessed as meeting criteria, being impaired, etc. Clearly, a single square meter where a grab sample is taken is not large enough to be an assessment unit. An assessment unit should consist of a definable segment, basin, or entire estuary. For example, a large complex estuary such as Puget Sound could be divided into its component inlet bays, canals, and passes. Many of the larger components could in turn be divided into segments.

Segmentation could be determined by some combination of mean salinity, water residence time, dominant substrate, or mean depth. For example, since estuarine fauna are determined by salinity, segmentation often corresponds to salinity zone (tidal fresh, oligohaline, mesohaline, polyhaline, and marine). Small estuaries, such as salt ponds in New England, could be single assessment units.

An assessment unit is the smallest spatial subdivision of an estuary that will be assessed; i.e., given a rating of good or poor. An assessment may be based on one or more sample units within an assessment unit. A sample unit (or sample site) is a site where an observation is made.

5.2.3 Specifying the Population and Sample Unit

Sampling is statistically expressed as a sample from a population of objects. Thompson (1992) suggested in some cases, the population is finite,

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countable, and easy to specify, (e.g., all persons in a city, where each person is a single member of the population). In estuaries, the population is often more difficult to specify and may be infinite, (e.g., the sediment of San Francisco Bay, where any location in the Bay defines a potential member of the population). Sampling units may be natural units (entire estuaries, cobbles on a beach), or they may be arbitrary (plot, quadrat, sampling gear area or volume) (Pielou 1977). Finite populations may be sampled with corresponding natural sample units, but often the sample unit (say, an estuary) is too large to measure in its entirety, and it must be characterized with one or more second stage samples of the sampling gear (bottles, benthic grabs, quadrats, etc.).

The objective of sampling is to best characterize individual sample units in order to estimate some attributes (e.g., nutrient concentrations, DO) and their statistical parameters (e.g., mean, median, variance, percentiles) of a population of sample units. The objective of the analysis is to be able to say something (estimate) about the population. Examples of sample units include:

- A point in an estuary (may be characterized by single or multiple sample device deployments). The population would then be all points in the estuary, an infinite population. This is the most common sample unit applied to estuarine assessments;

- A constant area, (e.g., square meter, hectare). The population would be an artificial one consisting of all square meters of estuarine surface area in an estuary, a state or a region;

- An estuary or a definable portion of the estuary as a single sample unit. Whole estuaries as sample units would only be used in very broad-scale regional assessments, as was done by EMAP-NC, for example, for small estuaries as a population (e.g., Strobel et al. 1995).

5.2.4 Sources of Variability

Variability of measurements has many possible sources, and the intent of many sampling designs is to minimize the variability due to uncontrolled or random effects, and conversely to be able to characterize the variability caused by experimental or class effects. For example, we may stratify estuarine waters by salinity and bottom substrate type (rocky, sandy, muddy). Typically, we stratify so that observations (sample units) from the same stratum will be more similar to each other than to sample units in other strata.

Environmental measures vary across different scales of space and time, and sampling design must consider the scales of variation. When sampling estuaries, measurements (say, benthic assemblages) are taken at single points in space and time (1 point along a transect in mid-summer). If the same measurement is made at a different place (littoral zone), embayment, or time (winter), the measured values will likely be different. A third component of variability is the ability to accurately measure the quantity interested in, which can be affected by sampling gear, instrumentation, errors in proper adherence to field and laboratory protocols, and the choice of methods used in making determinations.

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The basic rule of efficient sampling and measurement is to sample so as to minimize measurement errors; to maximize the components of variability that have influence on the central questions and reporting units; and to control other sources of variability that are not of interest, that is, to minimize their effects on the observations. Many locations are sampled in order to examine and characterize the variability due to different locations (the sampling unit). Each site is sampled in the same way, in the same place, and in the same time frame to minimize confounding variability.

In statistical terminology, there is a distinction between sampling error and measurement error that has little to do with actual errors in measurement. Sampling error is the error attributable to selecting a certain sample unit (e.g., an estuary or a location within an estuary) that may not be representative of the population of sample units. Statistical measurement error is the ability of the investigator to accurately characterize the sampling unit. Thus, measurement error includes components of natural spatial and temporal variability within the sample unit as well as actual errors of omission or commission by the investigator. Measurement error is minimized with methodological standardization: selection of cost-effective, low variability sampling methods, proper training of personnel, and quality assurance procedures to minimize methodological errors. In analytical laboratory procedures, measurement error is estimated by duplicate determinations on some subset of samples (but not necessarily all). Similarly, in field investigations, some subset of sample units should be measured more than once to estimate measurement error.

If the variance of individual measurements (measurement error) is unacceptably large; i.e., as large or larger than variance expected among sample units, then it is often necessary to alter the sampling protocol, usually by increasing sampling effort in some way, to further reduce the measurement error. Measurement error can be reduced by multiple observations at each sample unit, (e.g., multiple dredge casts at each sampling event, multiple observations in time during a growing season or index period, depth-integrated samples, or spatially integrated samples.

A less costly alternative to multiple measures in space is to make spatially composite determinations. In nutrient or chlorophyll determinations, a water column pumped sample, where the pump hose is lowered through the water column, is an example of a spatially composite determination. Spatial integration of an observation and compositing the material into a single sample is almost always more cost-effective than retaining separate, multiple observations. This is especially so for relatively costly laboratory analyses such as organic contaminants and benthic macroinvertebrates. Many estuarine programs have adopted sampling protocols consisting of multiple grabs at a site that are then composited into a single bucket for laboratory determinations (e.g., EMAP Near Coastal: 3 composited Van Veen grabs at each site; Holland 1990).

Statistical power is the ability of a given hypothesis test to detect an effect that actually exists, and must be considered when designing a

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sampling program (e.g., Peterman 1990, Fairweather 1991). The power of a test (1-β) is defined as the probability of correctly rejecting the null hypothesis (H_0) when H_0 is false; i.e. the probability of correctly finding a difference [impairment] when one exists. For a fixed confidence level (e.g., 90%), power can be increased by increasing the sample size or the number of replicates. To evaluate power and determine sampling effort, an ecologically meaningful amount of change in a variable must be set. See Chapter 12 for a discussion of statistical power, and examples.

Optimizing sampling design requires consideration of tradeoffs among the measures used, the effect size that is considered meaningful, desired power, desired confidence, and resources available for the sampling program. Every study requires some level of repeated measurement of sampling units to estimate precision and measurement error. Repeated measurement at 10% or more of sites is common among many monitoring programs.

5.2.5 Alternative Sampling Designs

Sampling design is the selection of a part of a population to observe the attributes of interest, in order to estimate the values of those attributes for the whole population. Classical sampling design makes assumptions about the variables of interest, in particular, it assumes that the values are fixed (but unknown) for each member of the population, until that member is observed (Thompson 1992). This assumption is perfectly reasonable for some variables, say, length, weight, and sex of members of an animal population, but it seems less reasonable for more dynamic variables such as nutrient concentrations, loadings, or chlorophyll concentrations of estuaries. Designs that assume that the observed variables are themselves random variables are model-based designs, where prior knowledge or assumptions (a model) are used to select sample units.

Probability-based designs (random sampling)

The most basic probability-based design is simple random sampling, where all possible sample units in the population have the same probability of being selected, that is, all possible combinations of n sample units have equal probability of selection from among the N units in the population. If the population N is finite and not excessively large, a list can be made of the N units, and a sample of n units is randomly selected from the list. This is termed *list frame sampling*. If the population is very large or infinite (such as locations in an estuary), one can select a set of n random (x,y) coordinates for the sample.

All sample combinations are equally likely in simple random sampling, thus there is no assurance that the sample actually selected will be representative of the population. Other unbiased sampling designs that attempt to acquire a more representative sample include stratified, systematic, multistage, and adaptive designs (Figure 5-2). In stratified sampling, the population is subdivided or partitioned into strata, and each stratum is sampled separately. Partitioning is typically done so as to make each stratum more homogeneous than the overall population. Systematic sampling is the systematic selection of every k^{th} unit of the population from one or

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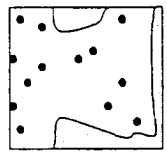
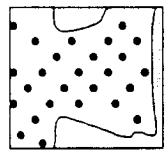
Sampling Methods

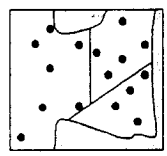
Simple Random: Samples are independently located at random



Systematic: Samples are located at regular intervals



Stratified: The study area is divided into nonoverlapping strata and samples are obtained from each



Multistage: Large primary units are selected which are then subsampled

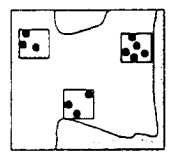


Figure 5-2
Description of various sampling methods. Adapted from USEPA 1992.

more randomly selected starting units, and ensures that samples are not clumped in one region of the sample space. Multistage sampling requires selection of a sample of large primary units, such as fields, hydrologic units, rectangles, or hexagons, and then selection of secondary sample units such as plots or estuaries within each primary unit in the first stage sample.

Estimation of statistical parameters requires weighting of the data with inclusion probabilities (the probability that a given unit of the population will be in the sample) specified in the sampling design. In simple random sampling, inclusion probabilities are by definition equal, and no corrections are necessary. Stratified sampling requires weighting by the inclusion probabilities of each stratum. Unbiased estimators have been developed for specific sampling designs, and can be found in sampling textbooks, such as Thompson (1992).

Model-based designs

Use of probability-based sampling designs may miss relationships among variables (models), especially if there is a regression-type relationship between an explanatory and a response variable. As an example, estimation of benthic response to discharge or outfalls requires a range of sites from those directly adjacent to the outfalls to those distant from, and presumably unaffected by, the outfalls (e.g. Warwick and Clarke 1991). A simple random sample of estuarine sites is not likely to capture the entire range, because there would be a large cluster of far sites, with few at high ends of the gradient. A simple random sample may therefore be highly inefficient with respect to models or specific hypotheses.

In model-based designs, sites are selected based on prior knowledge of auxiliary variables, such as estimated loading, depth, salinity, substrate

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type, etc. These designs preclude an unbiased estimate of the state of the estuaries, unless the model can be demonstrated to be robust and predictive, in which case the population value is predicted from the model and from prior knowledge of the auxiliary (predictive) variables. Selection of unimpacted reference sites is an example of a model-based design which cannot later be used for unbiased estimation of the biological status of estuaries. Ideally, it may be possible to specify a design that allows unbiased estimation of both population and model, with an appropriately stratified design. Statisticians should be consulted in developing the sample design for a biological criteria and monitoring program.

Selecting a Design

The selection of a station array for bioassessment will depend on the nature of the study and/or the desire to delineate the areal extent of impairment. A randomized station selection is most appropriate for environmental status and trends surveys such as conducted by EMAP. However, for specific management decision-making, pre-selected stations placed on a gradient such as distance from of a discharge (sometimes termed "nearfield/farfield") may be more appropriate. This method is a form of model-based design, and more accurately identifies suspected sources of impairment, assesses impacts and monitors recovery.

The number of stations to be incorporated in a study design is most heavily influenced by the available resources. A minimum of three control or reference sites is desired to provide some indication of background variability. The number of test sites may vary from one to several depending on the purpose of the study. The distance between stations could be decreased; i.e., number of stations increased to partially account for the inefficiency of some sampling gear or, conversely, the distance increased; i.e., number of stations decreased once the data have been evaluated.

Index Period

Most monitoring programs do not have the resources to characterize variability or to assess for all seasons. Sampling can be restricted to an index period when metrics are expected to show the greatest response to pollution stress and when within-season variability is small (Holland 1990). A decision must be made between selecting a sampling period that is representative of the biological community, or one that reflects the worst-case conditions for pollution stress. From the traditional perspective of evaluating pollution impacts in fresh water streams, summer-time low flow conditions are often chosen to assess effects from point source discharges. These flow conditions represent minimal effluent dilution in combination with the natural stressors of low water velocity and high temperature in those constrained environments. In contrast, the effects of nonpoint source pollution on the benthic community are often evaluated following periods of high flow since nonpoint source effects on aquatic communities are largely driven by runoff in the watershed. Estuaries and coastal waters accumulate materials from both nonpoint and point sources in a much more dynamic way and thereby confound the assessment so useful for streams.

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In bioassessment strategies involving infrequent sampling, the biologically-optimal period for sampling becomes a major consideration. Periods of instability in community structure, including recruitment of young, natural harsh environmental conditions, changes in food source, and migration of certain target populations are all considerations in conducting these biosurveys. The biologically-optimal period, usually mid-summer and sometimes mid-winter, avoids all of these elements and focuses on the time when communities are most stable. The resource manager or biologist will have to choose between these conditions, or select to cover both, depending on the needs of the study.

5.2.6 Optimizing Sampling

Ferraro et al. (1994, 1989) present a method for quantitatively evaluating the optimum macrobenthic sampling protocol, accounting for sampling unit area, sieve mesh size, and number of replicates (n). Their approach allows managers responsible for designing and implementing estuarine and coastal marine bioassessment programs to answer fundamental questions:

- How large should the sampling unit be?;
- What sieve mesh size should be used?;
- How many replicate samples should be taken?

The procedure calculates the "power-cost efficiency" (PCE), which incorporates both the number of samples (n), the cost (field collection effort and lab effort combined) and the expected statistical power for each alternative sampling scheme. See Chapter 12 for a more detailed discussion of statistical power. The various sampling schemes consist of different combinations of sampling gear, gear area, sieve mesh size, and number of replicates. The method allows determining the optimum among a set of sampling schemes for detecting differences in reference vs. impaired stations when the statistical model is a t-distribution for comparing two means. The optimum scheme can be defined as the least costly one capable of reliably (e.g., α = 0.5, 1-β = 0.95) detecting a desired difference in the means of a metric between two stations. The approach can be applied to each metric in a test set of metrics and the results aggregated to determine the optimum protocol.

There are four primary steps in assessing the PCE of a suite of alternative sampling schemes:

1. For each scheme, collect replicate samples at paired reference and impaired stations. The observed difference in metric values between the stations is operationally assumed to be the magnitude of the difference desired to be detected. Alternatively, a percentage of the median (e.g., 20%) for a given metric calculated across reference stations could be set as the magnitude of the difference to be detected. In either case, this difference, divided by the standard deviation, is the "effect size" (ES) of interest.

2. Assess the "cost" (c_i), in time or money, of each sampling scheme i at each station. The cost can include labor hours for sampling,

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sorting, taxonomic identification, and recording results.

3. Conduct statistical power analysis to determine the minimum number of replicate samples (n_i) needed to detect the ES with an acceptable probability of Type I (α) and Type II (β) error (e.g., $\alpha = \beta = 0.05$).

4. Calculate the power-cost efficiency (PCE) for each sampling scheme by:

$$PCE_i = (n \times c)_{min} / (n_i \times c_i)$$

where $(n \times c)_{min}$ = minimum value of $(n \times c)$ among the *i* sampling schemes. The reciprocal of PCE_i is the factor by which the optimal sampling scheme is more efficient than alternative scheme *i*. When PCE is determined for multiple metrics, the overall optimal sampling protocol may be defined as that which ranks highest in PCE for most metrics in the test set.

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Chapter 6

Water Column & Bottom Characteristics

Tiers 1-3 contain active survey and site sampling. Procedures for attaining water column and bottom characteristics are generally the same for each tier. The sampling however, occurs more often over the year. Differences are noted where applicable. Table 6-1 compares the level of effort for each tier. However, agencies will decide which components of each tier will be incorporated into their specific programs, then they will select the level of effort appropriate for their program.

6.1 Salinity, Temperature, Dissolved Oxygen, & pH

Salinity, conductivity, temperature, dissolved oxygen, and pH should be measured at each sampling station using a CTD meter equipped with DO and pH probes. Measurements should be made at 1-m intervals through the water column. In shallow, inshore waters, measurements should be taken at the top, middle, and bottom thirds of the depth. For Tier 3, in some southern waters that undergo significant diel temperature changes, it may be desirable to obtain 24-hour temperature profiles using recording equipment.

6.2 Secchi Depth

Secchi depth is usually measured at the deepest part of the transect or grid. Where the area is classified by depth, Secchi measurements should be made at each station. Readings are obtained with a 40-cm plastic or metal Secchi disk that is either white or is divided into black and white quadrants on a nonstretchable line that is calibrated in decimeters. The disk is lowered into the water until it disappears from view and the depth is recorded. The disk is then slowly raised to the point where it reappears, with the depth being recorded again. The mean of these two measurement is the Secchi depth. Observations are made from the shady side of the boat, without sunglasses, and as close as possible to the water to reduce glare.

6.3 Depth

Depth should be measured at each station using a calibrated depth sounder. Depth can be read off a meter block when sediment sampling by zeroing the block when the sampler is at the water surface. In shallow, inshore waters, a long stick or weighted line calibrated in decimeters may be used.

6.4 Sediment Grain Size

6.4.1 Estimation of "percent fines" (Tier 1)

Analysis of sediment grain size for Tier 1 assessments can be limited to determining the "percent fines" at each station. A rapid wet sieving technique used in Puget Sound (Eaton 1997) can serve as the basis for this characterization. Materials needed for the procedure include:

- standard testing sieve No. 230, 63-µm
- 50-ml plastic beaker (filled to the brim with sediments is about 79-ml)
- 100-ml plastic graduated cylinder
- water bottle(s) with small outlets

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Table 6-1. Water Column & Bottom Characteristics. "Addition" refers to added detail or intensities for a parameter initiated in an earlier tier.

Characteristic	Tier	Collection Method	Indicates
Salinity	1 2 3	-measure at each sampling station, CTD meter -continuous or 1-2-m intervals through water column -shallow/inshore -top, middle, bottom thirds of depth	Distribution of flora and fauna
Temperature	1 2	-measure at each station, CTD meter -1-2-m intervals through water column -shallow/inshore -top, middle, bottom thirds of depth	Rate of chemical reactions and biological processes
	3 addition	-some southern waters undergo significant diel changes, it may be desirable to obtain 24-hour temperature profiles	
Dissolved Oxygen	1	-measure at each station, CTD meter w/ DO probe -continuous or 1-2-m intervals through water column -shallow/inshore -top, middle, bottom thirds of depth	Possible reason for modified behavior, reduced abundance & productivity, adverse reproductive effects, and mortality
	2 addition	-measure early in morning at each station at minimum	
	3 addition	-collect along a depth profile from surface to within 1-m of bottom at 1-2-m intervals -in cases of hypoxic site: recording DO meters may be deployed (EMAP - Louisianian Province, Engle et al 1994)	Resource agency may determine need for more detailed information to diagnose sources & causes of impairment
pH	1 2 3	-CTD w/ pH probe -1-2-m intervals -top, middle, bottom thirds of depth	Chemical condition, pollutant input, high concentrations of phytoplankton
Secchi Depth (Turbidity)	1 2 3	-deepest part of transect/grid -if area classified by depth, measure at each station -See Section 6.2 for complete procedure	Reduction of light penetration, deposition of mud and silt, possible contaminated sediment "hot spots"
Depth	1 2 3	-each station w/ calibrated depth sounder -read off meter block when sediment sampling -shallow/inshore waters: long stick or weighted line calibrated in decimals	Depth at sampling station, possible dredging or sediment loading
Nutrients	2 3	-collected w/ bottle samplers or pump -see Section 6.8 for complete procedure	Nutrient loading
Acid Volatile Sulfides	3	-each station during index period, and any other sampling visits through year -once accurate AVS exists for each station, analytes only performed once per year (during index period)	Bottom characteristics, detailed purposes in Section 3.5.4
Water Column Contaminants	3	Choose One: -USEPA's list of Priority Pollutants, Hazardous Substance, or Target Compound/Analytes -same compounds targeted in EMAP-E (Table 3-1) -develop own list (see Section 6.10 for more detail)	Trace distribution of contaminants from a source or to ID potential sources

Table 6-1 (Cont'd). Water Column & Bottom Characteristics. "Addition" refers to added detail or intensities for a parameter initiated in an earlier tier.

Characteristic	Tier	Collection Method	Indicates
Sediment Grain Size	1	-determine "percent fines" at each station, see Section 6.4.1 for complete procedure	Spatial and temporal changes of the benthic habitat, evaluate condition of benthic habitats
	2	-see Section 6.4.2 for complete procedure	Determine extent or recovery from environmental perturbations
	3		Assist in providing early warnings of potential impacts to the estuarine ecosystem
Total Organic Carbon	2	-see Section 6.9 for complete procedure	Provide information regarding sediment organic content (possibly influenced by sewage outfalls)
	3 addition	-measure additional sediment analytes	Examine potential influences of outfalls, ID potential contaminant "hot spots"
RPD Layer Depth	1	-vertical bisection, distance from sediment surface to a noticeable change in color from brownish (oxidizing conditions) to gray (reducing conditions)	Note presence/absence of benthos; learn about life history, taxa abundance, & major taxa biomass distribution; more large, deep dwelling species="healthy" system
Total Volatile Sulfides	1	-deepest section along transect/grid -see Standard Methods (APHA 1992) for sampling & analytical methods	Sediment and carbon content
Sediment Contaminants	1	-conducted at outset of survey	Positive=severe impacts influence spatial sampling design, causal investigations Negative=subsequently collected biological info. essential to ID other, possibly more subtle stresses Provide insight on limiting factors in benthic community
	2	*like TOC, if toxicity tests are initially negative, no need to repeat annually unless biological data from infauna indicate otherwise	
	3	Choose from three approaches: -based on EPA's contaminant lists -NOAA NS&T suite of contaminants (used by EMAP) -targeted list *see Section 6.12 for complete procedure and rationale	

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- small stainless or plastic spatula
- stainless butter knife
- hose with nozzle (if running water is available).

Detailed directions for performing this wet-sieving technique are as follows:

Fill a 50-ml plastic beaker to the brim with the sediment to be analyzed. The capacity of the completely filled beaker can be measured using water and the 100-ml graduated cylinder. Clean away any sediment that might adhere to the outside of the beaker. Carefully wash this sediment through a 63-µm standard sieve (USA standard testing sieve No. 230) with stainless steel mesh. The sieve itself is about 9" in diameter with a 2" stainless lip. Be careful not to overflow the sieve with rinsing water. It may be easier to wash half of the sediment through at a time. If running water is available, use a small brass nozzle on the end of the hose with very low water pressure when washing the sediment, otherwise the sediment will need to be washed using the water bottle. If there are occasional large worm tubes or shells, these are discarded and replaced with an approximately equal volume of sediment. The sediment remaining on the sieve is the coarse-grained fraction. This is washed to one side of the sieve, and then carefully placed into the plastic 100-ml graduated cylinder with a stainless steel butter knife, and finally with the small stainless spatula. The water bottle is then used to wash any remaining sediment directly into the graduated cylinder, and to wash down the sides of the cylinder. Let the sediment-water mixture settle in the 100-ml graduated cylinder for approximately 5 minutes until the supernatant water is clear. This may take longer for very fine-grained sediments. Note the volume of the coarse-grained fraction which remains after sieving. This can be divided by the original volume to obtain the percentage of the coarse fraction. The standard usage, however, is for percent fine-grained fraction or "**percent fines**". This is calculated by subtracting the volume of sediment remaining in the cylinder (ml of coarse-grained fraction) from the original volume, and dividing this number (ml of fine-grained) by the original volume to obtain the percent fines.

6.4.2 Sediment Grain Size (Tiers 2 and 3)

Additional grain size data for Tier 2 and Tier 3 assessments should include determination of the size distribution using a standard graded sieve series. This analysis should be performed for a sediment sample collected at each sampling station. In the early years of the assessment program, this analysis should be performed for each sampling period. When an accurate sediment characterization exists for the area of each station, sediment grain size analysis could be performed only annually or biennially (on samples collected in the index period), unless the agency believed that sediment conditions at a site may have changed. This could occur, for example, following a major storm. Buller and McManus (1979) provide a review of the methodological and statistical analysis of sediment samples. If seasonal variations in grain size are exhibited, it is recommended that direct comparisons between samples collected during different seasons be avoided. Studies investigating interannual variation in the percent composition of grain sizes should be conducted during the same season (preferably the same month) each year. Furthermore, it is recommended that grain size be sampled when contaminant concentrations are expected to be at their highest level to evaluate worst-case scenarios.

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6.5 RPD Layer Depth

The concept behind using the depth distribution of benthic macroinvertebrates is based on the premise that "healthy" benthic communities in fine sediments in meso- and polyhaline waters consist of relatively large, deep dwelling species; while impaired areas will have fewer of these organisms. The depth distribution of benthic infauna in sediments integrates functional parameters such as life history, taxa abundance, and major taxa biomass distribution.

6.6 Total Volatile Solids

Total volatile solids (TVS) is the Tier 1 indicator for sediment carbon content. TVS should be determined for the deepest station along each transect or grid, based on the assumption that deeper stations will represent sinks for organic carbon in the sediments. Sampling and analytical methods are discussed in Standard Methods (APHA 1992).

6.7 Sediment Contaminant Toxicity

Sediment toxicity testing is a diagnostic indicator for Tier 3. When results are positive for a station, severe impacts at a known locality will influence spatial sampling design and causal investigations. Where toxicity test results are negative throughout the set of stations sampled, subsequently collected biological information is essential to identify other, possibly more subtle stresses on the system.

6.7.1 10-day Static Sediment Toxicity Test with Marine and Estuarine Amphipods

ASTM (1998a) and USEPA (1994b) developed procedures that measure short term adverse effects of potentially contaminated sediment, or of a test material experimentally added to sediment, on marine or estuarine infaunal amphipods during static 10-day exposures for the following species: *Rhepoxynius abronius, Eohaustorius estuaris, Ampelisca abdita, Grandidierella japonica,* and *Leptocheirus plumulosus.* The amphipod *Corophium insidiosum* has also been used in standard testing (Reish and Lemay 1988). Solid phase tests use overlying water in aerated 1-L glass test chambers. Mortality and sublethal effects such as growth, emergence of adults, and inability to bury in clean sediment are determined after exposure of a specific number of amphipods (usually 20) to the test sediment. Response of the amphipods to the test sediment is compared with the response observed in control or reference sediment. The negative control sediment is used to provide a measure of the acceptability of the test by providing evidence of the health and relative quality of the test organisms, the suitability of the overlying water, and test conditions and handling procedures (ASTM 1998b, USEPA 1994b). The reference sediment, which is similar in physical characteristics to the test sediments and typically collected from a similar location, is used as the basis for interpreting data obtained from the test sediments (ASTM 1998b).

The toxicity of field-collected sediments may be assessed by either (a) testing the whole sediment and testing for significant differences in responses between reference or control and test sediment exposed animals or (b) testing dilutions of a test sediment with clean sediment to obtain an LC_{50} or other effect concentration, for survival, reburial success, or growth (ASTM 1998b, Nelson et al. 1993, Swartz et al. 1995).

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6.7.2 10-day Static Sediment Toxicity Test with Marine and Estuarine Polychaetous Annelids

Marine or estuarine infaunal polychaetes are used in whole sediment tests during 10-day or 20- to 28-day exposures to determine adverse effects of potentially contaminated sediment, or of a test material added experimentally to sediment. Polychaete species include *Neanthes virens* for the 10-day and *Neanthes arenaceodentata* for the 10-day and 20- to 28-day tests (ASTM 1998c). Other polychaete species that have been used in similar sediment testing include *Capitella capitata, Ophrotrocha diadema,* and *Ctenodrilus serratus* (Reish and Lemay 1988). The 10-day test measures effects of contaminated sediment on polychaete survival. The 20- to 28-day test determines effects of contaminated sediment on polychaete survival and growth. If smaller species are used, such as *N. arenaceodentata,* five worms are placed in a 1-L glass test chamber with a minimum sediment depth of 2- to 3-cm and the overlying water is aerated. Either young adults or recently emerged juvenile (2- to 3-weeks post-emergence) worms are used in the 10-day test; only recently emerged (2- to 3-weeks) juveniles are used in the 20- to 28-day test. Survival of worms exposed to the test sediment is compared with the survival in a negative control or reference sediment in either test. If larger species are used, such as *N. virens,* ten worms are placed in a glass aquaria (4- to 37-L) with a minimum sediment depth of 10-cm and the overlying water is aerated.

The percent survival of polychaetes exposed to field-collected sediment is compared to those exposed to a negative control or reference sediment in 10-day tests. Survival and body weight of surviving animals is compared to those exposed to negative control or reference sediment in 20- to 28-day tests. The toxicity of field sediments may also be assessed by testing dilutions of highly toxic test sediments with clean sediments to obtain either an LC_{50} or other effect concentration of the material.

6.7.3 Static Acute Toxicity Tests with Echinoid Embryos

Echinoderm embryos and larval form sea urchins (*Strongylocentrotus purpuratus* and *Strongylocentrotus droebachiensis*) and sand dollars (*Arbacia punctulata, Lytechinus pictus,* and *Dendraster excentricus*) have been used in marine sediment interstitial (pore) water tests (ASTM 1998a). Interstitial water from marine sediments is isolated using either *in-situ* peepers (Sarda and Burton 1995, Brumbaugh et al. 1994, Bufflap and Allen 1995), suction in the field (Watson and Frickers 1990), laboratory centrifugation (Ankley et al. 1991, Burgess et al. 1993, Kemble et al. 1994, ASTM 1998b), or sediment squeezing (Long et al. 1990). Embryos are obtained by inducing adults to spawn, using either physical (e.g., electric stimuli) or chemical (injection of potassium chloride) means, and then combining gametes.

Embryos are exposed to the test pore water and controls (culture water) for 48- to 96-hours, depending on the species and test temperature. The test measures the proportion of embryos or larvae that develop into normal pluteus larvae. Pore waters can be tested "whole"; i.e., undiluted, and organism responses expressed in terms of a significant difference between controls and test waters. Alternatively, pore water samples can be diluted with known, clean culture water and the results expressed as an LC_{50} or other

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effect concentration with confidence limits.

6.7.4 Toxicity Tests Using Marine Bivalves

Juveniles of the marine bivalve species, *Mulinia lateralis*, have been used in whole sediment tests (Burgess and Morrison 1994). Juveniles are exposed for 7-days to determine adverse effects of potentially contaminated sediment, or of a test material added to sediment. Bivalve responses measured include survival and growth, (total organism dry weight). Ten juvenile bivalves (four weeks old) are placed into six replicate chambers per sediment or treatment. The sediment exposure chambers are prepared by placing approximately 1.0-cm deep sediment into 150-ml dishes, followed by the addition of 100-ml filtered 30-gkg^{-1} seawater. Upon initiation of the test a subsample of organisms are set aside for determination of initial juvenile weights. Bivalve survival in test chambers is compared to survival of bivalves in the negative control or reference sediment. Dry weight of the surviving organisms in test chambers is compared with dry weight of surviving organisms in the reference sediment, and to the dry weight of the subsample set aside at the initiation of the test to determine growth.

Similar to echinoderm testing summarized in Section 6.7.3, bivalve larvae have also been used in sediment pore water and sediment elutriate toxicity tests. Species used include *Crassostrea gigas* and *Mytilus edulis* (PSEP 1995). Bivalve larvae are obtained from laboratory-cultured adult brood stock, which are induced to spawn. Developing embryos are exposed to the pore water or elutriate at 20°C for 48-60-hours using static-test conditions. At test termination, subsamples of the larvae are examined and the percentages of mortality and abnormal survivors are determined and analyzed.

6.8 Nutrients (Tiers 2&3)

Water column samples for nutrient analysis can be collected using bottle samplers such as Kemmerer, Van Dorn, Niskin, or Nansen samplers. A pump may be used as an alternative sampling device. In shallow water less than 2-m depth, a mid-depth sample at each station should be obtained for nutrient analysis. In waters greater than 2-m depth, samples should be collected at each station at 1-m below the surface, 1-m above the bottom, 1-m above the pycnocline, 1-m below the pycnocline, or at mid-depth. Analytical methods for NH_4-N, NO_3-N, NO_2-N, Kjeldahl nitrogen, total N, and total and reactive P; i.e., ortho-P, are presented in APHA (1992) and USEPA (1994c). These nutrient analyses will help identify eutrophication factors affecting biocriteria development, as well as supplement the USEPA's nutrient criteria initiatives so that multiple objectives can be accomplished at once.

6.9 Total Organic Carbon (Tiers 2&3)

In Tier 2, the primary purpose of measuring total organic carbon (TOC) is to provide information regarding sediment organic content, which might be influenced by sewage outfalls containing high organic levels. As noted in Chapter 3, TOC in the sediment is an important analyte for the purpose of evaluating the bioavailability of organic pollutants and metals adsorbed by sediments or contained in sediment porewater. Data on sediment TOC collected in this tier can be used to examine potential influences of outfalls in addition to potential sediment contaminant "hot spots" that can be

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assessed in Tier 3 with the measurement of additional sediment analytes.

Standard methods for TOC analysis are presented in APHA (1992). In the early years of the assessment program, TOC analysis should be performed for each station in each sampling period. Once the resource agency is confident that an accurate characterization of sediment TOC exists for each station, the analysis could be performed only once every two or more years (on samples collected in the index period), unless stations that appear to be influenced by organic input (e.g., sewage outfalls) are identified. In this case, TOC analysis should continue to be performed for each sampling period for these stations.

6.10 Water Column Contaminants (Tier 3)

Water column contaminants such as organic compounds (e.g., herbicides, pesticides, hydrocarbons) and metals may be important indicators of sources and causes of impairment to biological assemblages in estuaries and coastal marine waters. Decisions on which chemicals to include in Tier 3 assessments can be difficult. Three approaches to selecting contaminants might be useful. One approach would be to analyze for all chemicals listed on USEPA's Priority Pollutant, Hazardous Substance, or Target Compound/ Analyte Lists. A second approach would be to analyze for the same compounds targeted in the EMAP-Estuaries program (refer to Table 3-1). A third approach would be to develop a targeted list. In this latter approach, the historical information from Tier 0 and subsequent follow-up inquiries of land use in the suspect area could point to common pesticides, herbicides, or industrial products or byproducts that could form the basis of a select list of contaminants to analyze. Sources for this information also include NPDES permit records and discharger toxicity test results. In any case, three replicate water samples should be collected at each sampling station within an appropriate index period and on at least three other visits during the year to capture temporal variations in contaminant concentrations. Historic water contaminant data, plus data collected in this tier, can be used by the state to determine a more limited list of analytes for subsequent years of the assessment and biocriteria program.

The same type of sampling bottle used to collect water samples for nutrient analysis may be used for contaminant samples. USEPA (1992) and APHA (1992) contain detailed information on analytical methods.

6.11 Acid Volatile Sulfides (Tier 3)

Details of the purposes for measuring acid volatile sulfides (AVS) present in bottom sediments are provided in Section 3.5.4. Given the diagnostic intent of a Tier 3 assessment, it is important to include this analyte in determinations of bottom characteristics only if metals are suspected as a cause of biological degradation. Allen et al. (1993) discuss analytical methods for this parameter. AVS measurements should be made on sediment samples collected at each station during an appropriate index period and any other sampling visits made throughout the year. Once the resource agency is confident that an accurate characterization of sediment AVS exists for each station, the analytes should be performed only once per year (on samples collected in the index period).

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6.12 Sediment Contaminants

As with water column contaminants, three approaches to selecting analytes could be used: (1) a full scan based on USEPA's contaminant lists; (2) the NOAA National Status and Trends suite of contaminants used by the EMAP program (refer to Table 3-1); or (3) a targeted list.

In this latter approach, the historical information from Tier 0 and subsequent follow-up inquiries of land use in the suspect area could point to common pesticides, herbicides, or industrial products and byproducts that could form the basis of a select list of contaminants to analyze. In addition to sampling organisms for contaminants, sediment samples should be collected from the device used for sampling benthic infauna. The surface sediment (top 2-cm) should be removed from replicate grab samples and composited. During collection, care should be taken to avoid collecting material from the edge of grabs and to use only samples that have undisturbed sediment surfaces. The composite sample should be homogenized, and a subsample taken for measurement of contaminant concentrations. Analytical methods are discussed in APHA (1992) and USEPA (1992).

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ATTACHMENT B

RECEIVING WATERS MONITORING AND REPORTING PROGRAM FOR ORDER NO. 2001-01

Countywide to Watershed Based Monitoring and Reporting Program
The primary objectives of the Receiving Waters Monitoring and Reporting Program include, but are not limited to: 1) assessing compliance with Order No. 2001-01; 2) measuring the effectiveness of Urban Runoff Management Plans; 3) assessing the chemical, physical, and biological impacts to receiving waters resulting from urban runoff; and 4) assessing the overall health and evaluating long-term trends in receiving water quality.

Like Order No. 2001-01 in general, the monitoring requirements below are intended to transition during the five-year permit period from a countywide approach to a watershed based approach. During the first two reporting periods[1] of this Order, this monitoring program shall be conducted and reported on the same countywide basis as previously conducted under Order No. 90-42. Specifically, all monitoring shall be conducted jointly by all Copermittees under a single contractor with countywide coordination.

Beginning with the third monitoring period of this Order (unless otherwise directed by the SDRWQCB Executive Officer) the design of the monitoring program will shift to a watershed based approach. The monitoring program design, implementation, analysis, assessment, and reporting shall be conducted on a watershed basis for each of the nine hydrologic units. Monitoring results shall be assessed and reported on a watershed basis as a single report by the Copermittees consisting of one common section and nine watershed sections. Monitoring, analysis, assessment, and reporting shall satisfy the requirements of specified below for each watershed as applicable.

Order No. 2001-01 may be modified by the SDRWQCB Executive Officer without further public notice to direct the Copermittees to participate in comprehensive regional monitoring activities in the Southern California Bight in lieu of specific Order 2001-01 receiving waters monitoring requirements during the term of this Order.

I. Previous Monitoring and Future Recommendations Report

The Copermittees shall collaborate to develop a "Previous Monitoring and Future Recommendations Report" that summarizes all previous wet weather monitoring results and recommends future monitoring activities including the possibility of participating in coordinated comprehensive regional monitoring in the Southern California Bight. The Principal Permittee shall be responsible for the writing of the report and submittal to the SDRWQCB within **180 days** of adoption of this Order. At a minimum, the report shall:

A. Summarize the cumulative findings of all previous wet weather monitoring;
B. Identify detectable trends in water quality data and receiving water quality, based on the cumulative previous wet weather monitoring findings;
C. Interpret the cumulative previous wet weather monitoring findings;
D. Draw conclusions regarding the cumulative previous wet weather monitoring findings;
E. Provide recommendations for future monitoring activities; and
F. Include an executive summary, introduction, conclusion, and summary of recommendations.

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[1] A reporting period is defined as October 1st to September 30th of any year. The first reporting period under this Order is October 1, 2001 to September 30, 2002.

II. Receiving Waters Monitoring Program - - Year Round

Utilizing the findings of the "Previous Monitoring and Future Recommendations Report" discussed above, the Copermittees shall collaborate to develop, submit, conduct, and report on a year round countywide or watershed based Receiving Waters Monitoring Program[2]. The goals of both the countywide and watershed based Receiving Waters Monitoring Program shall be clearly stated. The Receiving Waters Monitoring Program goals shall focus on assessing compliance with this Order, achieving water quality objectives, protecting beneficial uses, and assessing the overall health and long-term water quality trends of receiving waters. For purposes of conducting the countywide or watershed based Receiving Waters Monitoring Program, the Copermittees are encouraged to collaborate with other agencies conducting similar monitoring, such as the Southern California Coastal Water Research Project (SCCWRP), the California Department of Fish and Game, or other municipalities in Southern California. Implementation of the countywide or watershed based Receiving Waters Monitoring Program shall begin within **180 days** of adoption of this Order. The countywide or watershed based Receiving Waters Monitoring Program shall include, at a minimum, the following components:

A. Urban Stream Bioassessment Monitoring
B. Long-term Mass Loading Monitoring
C. Coastal Storm Drain Outfall Monitoring
D. Ambient Bay, Lagoon, and Coastal Receiving Water Monitoring
E. Toxic Hot Spots Monitoring in San Diego Bay

A. <u>Urban Stream Bioassessment Monitoring</u>

1. The Copermittees shall collaborate to develop and implement an urban stream bioassessment monitoring program. At a minimum, the program shall consist of station identification, sampling, monitoring, and analysis of data for 20 bioassessment stations in order to determine the biological and physical integrity of urban streams within the County of San Diego. In addition to the urban stream bioassessment stations, three reference bioassessment stations shall be identified, sampled, monitored, and analyzed. The selection, sampling, monitoring, and analysis of bioassessment stations shall meet the following requirements:

   a. Each urban stream bioassessment station shall be selected using the following criteria. Each urban stream bioassessment station shall:
      (1) be located within the jurisdiction of a Copermittee; or
      (2) be located within one of the nine watersheds specified in Section J, Table 4 of this Order; and
      (3) be representative of urban stream conditions within one of the nine watersheds specified in Section J, Table 4 of this Order; and
      (4) meet the physical criteria of the California Stream Bioassessment Procedure[3]; and
      (5) to the extent feasible, coincide with the location of an already existing monitoring station used by the California Department of Fish and Game in the conduct of the SDRWQCB's Ambient Bioassessment Program.

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[2] During the first two years, monitoring and reporting will be conducted and reported on a countywide basis. Beginning in the third monitoring period of Order 2001-01, the monitoring and reporting program will shift to a watershed based approach.

[3] California Stream Bioassessment Procedure (Protocol Brief for Biological and Physical/Habitat Assessment in Wadeable Streams), California Department of Fish and Game – Aquatic Bioassessment Laboratory, May 1999.

b. Each bioassessment station shall be monitored twice annually, in May and October of each year, beginning in May 2001. A minimum of three replicate samples shall be collected at each station during each sampling event.

c. Sampling, laboratory, quality assurance, and analysis procedures shall follow the standardized procedures set forth in the California Department of Fish and Game's California Stream Bioassessment Procedure (CSBP). Analysis procedures shall include comparison between station mean values for various biological metrics. Sampling, laboratory, quality assurance, and analytical procedures shall follow the standardized "Non-Point Source Bioassessment Sampling Procedures" for professional bioassessment set forth in the CSBP. In the event that the CSBP "Point-Source Professional Bioassessment Procedure" is performed in place of the "Non Point Source Bioassessment Sampling Procedure," justification and documentation of the procedure shall be submitted with the report. Results of the Urban Stream Bioassessment Monitoring shall be reported annually as part of the overall Receiving Waters Monitoring and Reporting Program for Order No. 2001-01. Reporting of the bioassessment data shall follow the format of the San Diego Regional Water Quality Control Board 1999 Biological Assessment Annual Report[4]. The report shall include:

(1) All physical, chemical and biological data collected in the assessment;
(2) Photographic documentation of assessment and reference stations;
(3) Documentation of quality assurance and control procedures;
(4) Analysis that shall include calculation of the metrics used in both the CSBP and the 1999 Annual Report.
(5) The report shall provide interpretation for comparisons of mean biological and habitat assessment metric values between assessment and reference stations.
(6) Utilize a regional index of biological integrity as part of the analysis.
(7) Electronic data formatted to California Department of Fish and Game Aquatic Bioassessment Laboratory specifications for inclusion in the Statewide Access Bioassessment database.

d. A professional environmental laboratory shall perform all sampling, laboratory, quality assurance, and analytical procedures. While valuable, data collected by volunteer monitoring organizations shall not be submitted in place of professional assessments.

e. Reference stations shall be selected following the recommendations in the 1999 Annual Report, Hughes (1995)[5] and Barbour et. al. (1999)[6]. Reference stations shall be evaluated annually by the Copermittees for suitability and the results included in the annual report. New reference stations will be selected as needed by the Copermittees.

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4 San Diego Regional Water Quality Control Board ,1999 Biological Assessment Annual Report. A Water Quality Inventory Series: Biological and Physical/Habitat Assessment of California Water Bodies. California Department of Fish and Game Office of Spill Prevention and Response, Water Pollution Control Laboratory. December 1999.

5 Hughes, R. M. (1995) Defining Acceptable Biological Status by Comparing with Reference Conditions in Biological Assessment and Criteria: Tools for Water Resource Planning and Decision Making, Wayne S. Davis and Thomas P. Simon eds. Lewis Publishers, Boca Raton, LA.

6 Barbour, M.T. , J Gerritsen, B.D. Synder, and J.B. Stribling (1999) Rapid Bioassessment Protocols For Use in Streams and Wadeable Rivers: Periphyton, Benthic Macroinvertebrates, and Fish. Second Edition. EPA 841-B-99-002

2. The Copermittees shall design and implement a program to conduct standardized toxicity testing at urban stream bioassessment stations where the bioassessment data indicates significant impairment. When findings indicate the presence of toxicity, a Toxicity Identification Evaluation (TIE) shall be conducted to determine the cause(s) of the toxicity.

B. Long-term Mass Loading Monitoring

For purposes of evaluating long-term trends, the Copermittees shall continue to monitor the five existing long-term mass loading stations as specified in Monitoring and Reporting Program No. 95-76 and amended by Technical Change Order Nos. 1-4. When findings indicate the presence of toxicity, a Toxicity Identification Evaluation (TIE) shall be conducted to determine the cause(s) of the toxicity.

C. Coastal Storm Drain Outfall Monitoring

The Copermittees shall collaborate to develop and implement a monitoring program for discharges of urban runoff from coastal storm drain outfalls. The program shall meet the following requirements:

1. The program shall include rationale and criteria for selection of storm drain outfalls to be monitored.

2. The program shall include collection of samples for analysis of total coliform, fecal coliform, and enterococci, in addition to any other indicators or pathogens identified by the Copermittees.

3. Samples shall be collected at both the storm drain outfall and in the surf zone (at ankle to knee water depths) directly in front of the outfall.

4. Samples shall be collected during both dry and wet weather periods.

5. Exceedances of public health standards for bacteria must be reported to the County Department of Public Health as soon as possible by the Copermittees.

D. Ambient Bay, Lagoon, and Coastal Receiving Water Monitoring

The Copermittees shall collaborate to develop and implement a program to assess the overall health of the receiving water and monitor the impact of urban runoff on ambient receiving water quality. This monitoring shall including San Diego Bay, Mission Bay, Oceanside Harbor, the Pacific Ocean coastline, coastal lagoons and estuaries, and all Clean Water Act section 303(d) water bodies or other environmentally sensitive areas as defined in F.1.b(2)(a)vii of this Order .

E. Toxic Hot Spots Monitoring in San Diego Bay

The Copermittees shall collaborate to develop and implement a program to assess the relative contribution of urban runoff on Toxic Hot Spots in San Diego Bay.

III. Submittal of Receiving Waters Monitoring Program Document

The Principal Permittee shall submit to the SDRWQCB the countywide or watershed based Receiving Waters Monitoring Program within **180 days** of adoption of this Order. The regional or watershed based Receiving Waters Monitoring Program shall describe how the Copermittees will meet the requirements of the components outlined in Section II of this Attachment.

IV. Submittal of Receiving Waters Monitoring Annual Reports

The Principal Permittee shall submit the Receiving Waters Monitoring Annual Report to the SDRWQCB on January 31 of each year, beginning on January 31, 2003.

V. Monitoring Annual Report Requirements

A. Monitoring reports shall provide the data/results, methods of evaluating the data, graphical summaries of the data, and an explanation/discussion of the data for each monitoring program component listed above.

B. Monitoring reports shall include an analysis of the findings of each monitoring program component listed above. The analysis shall identify and prioritize water quality problems. Based on the identification and prioritization of water quality problems, the analysis shall identify potential sources of the problems, and recommend future monitoring and BMP implementation measures for identifying and addressing the sources. The analysis shall also include an evaluation of the effectiveness of existing control measures.

C. Monitoring reports shall include identification and analysis of any long-term trends in storm water or receiving water quality.

D. Monitoring reports shall provide an estimation of total pollutant loads (wet weather loads plus dry weather loads) due to urban runoff for each of the watersheds specified in Section J, Table 4 of Order No. 2001-01.

E. Monitoring reports shall for each monitoring program component listed above, include an assessment of compliance with applicable water quality standards.

F. All monitoring reports shall use a standard report format and shall include the following:

1. A stand alone comprehensive executive summary addressing all sections of the monitoring report;
2. Comprehensive interpretations and conclusions; and
3. Recommendations for future actions.

G. All monitoring reports submitted to the Principal Permittee or the SDRWQCB shall contain the certified perjury statement described in Standard Reporting Requirements in Attachment C section B.10.d.

H. All monitoring reports shall be reviewed prior to submittal to the SDRWQCB by a committee (consisting of no less than three members). All review comments shall also be submitted to the SDRWQCB.

I. All monitoring reports shall be submitted in both electronic and paper formats.

J. All monitoring reports shall describe monitoring station locations by latitude and longitude coordinates, frequency of sampling, quality assurance/quality control procedures and sampling and analysis protocols.

K. Monitoring programs and reports shall comply with Section VI of Attachment B, as well as Attachment C.

VI. Standard Monitoring Requirements

A. All monitoring activities shall meet the following requirements:

1. Monitoring and Records [40 CFR 122.41(j)(1)]

Samples and measurements taken for the purpose of monitoring shall be representative of the monitored activity.

2. Monitoring and Records [40 CFR 122.41(j)(2)] [California Water Code § 13383(a)]

The discharger shall retain records of all monitoring information, including all calibration and maintenance records and all original strip chart recordings for continuous monitoring instrumentation, copies of all reports required by this Order, and records of all data used to complete the application for this permit, for a period of at least three years from the date of the sample, measurement, report or application. This period may be extended by request of the SDRWQCB at any time.

3. Monitoring and Records [40 CFR 122.21(j)(3)]

Records of monitoring information shall include the information requested in Attachment B and the following:

a. The date, exact place, and time of sampling or measurements;
b. The individual(s) who performed the sampling or measurements;
c. The date(s) analyses were performed;
d. The individual(s) who performed the analyses;
e. The analytical techniques or methods used; and
f. The results of such analyses.

4. Monitoring and Records [40 CFR 122.21(j)(4)]

Monitoring results must be conducted according to test procedures approved under 40 CFR part 136 unless other test procedures have been specified in this Order.

5. Monitoring and Records [40 CFR 122.21(j)(5)]

The Clean Water Act provides that any person who falsifies, tampers with, or knowingly renders inaccurate any monitoring device or method required to be maintained under this Order shall, upon conviction, be punished by a fine of not more than $10,000, or by imprisonment for not more than two years, or both. If a conviction of a person is for a violation committed after a first conviction of such person under this paragraph, punishment is a fine of not more than $20,000 per day of violation, or by imprisonment of not more than four years, or both.

6. Monitoring and Records [40 CFR 122.41(k)(2)]

The Clean Water Act provides that any person who knowingly makes any false statement, representation, or certification in any record or other document submitted or required to be maintained under this permit, including monitoring reports or reports of compliance or non-compliance shall, upon conviction, be punished by a fine of not more than $10,000 per violation, or by imprisonment for not more than six months per violation, or by both.

7. Monitoring Reports [40 CFR 122.41(l)(4)

Monitoring results shall be reported at the intervals specified elsewhere in this Order.

8. Monitoring Reports [40 CFR 122.41(l)(4)(ii)]

If the discharger monitors any pollutant more frequently than required by the permit using test procedures approved under 40 CFR part 136, unless otherwise specified in the Order, the results of this monitoring shall be included in the calculation and reporting of the data submitted in the reports requested by the SDRWQCB.

9. Monitoring Reports [40 CFR 122.41(l)(4)(iii)]

Calculations for all limitations which require averaging of measurements shall utilize an arithmetic mean unless otherwise specified by the SDRWQCB in the Order.

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ATTACHMENT B

RECEIVING WATERS MONITORING AND REPORTING PROGRAM
FOR
ORDER NO. 2001-01

Countywide to Watershed Based Monitoring and Reporting Program
The primary objectives of the Receiving Waters Monitoring and Reporting Program include, but are not limited to: 1) assessing compliance with Order No. 2001-01; 2) measuring the effectiveness of Urban Runoff Management Plans; 3) assessing the chemical, physical, and biological impacts to receiving waters resulting from urban runoff; and 4) assessing the overall health and evaluating long-term trends in receiving water quality.

Like Order No. 2001-01 in general, the monitoring requirements below are intended to transition during the five-year permit period from a countywide approach to a watershed based approach. During the first two reporting periods[1] of this Order, this monitoring program shall be conducted and reported on the same countywide basis as previously conducted under Order No. 90-42. Specifically, all monitoring shall be conducted jointly by all Copermittees under a single contractor with countywide coordination.

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Order No. 2001-01 may be modified by the SDRWQCB Executive Officer without further public notice to direct the Copermittees to participate in comprehensive regional monitoring activities in the Southern California Bight in lieu of specific Order 2001-01 receiving waters monitoring requirements during the term of this Order.

I. Previous Monitoring and Future Recommendations Report

The Copermittees shall collaborate to develop a "Previous Monitoring and Future Recommendations Report" that summarizes all previous wet weather monitoring results and recommends future monitoring activities including the possibility of participating in coordinated comprehensive regional monitoring in the Southern California Bight. The Principal Permittee shall be responsible for the writing of the report and submittal to the SDRWQCB within **180 days** of adoption of this Order. At a minimum, the report shall:

A. Summarize the cumulative findings of all previous wet weather monitoring;
B. Identify detectable trends in water quality data and receiving water quality, based on the cumulative previous wet weather monitoring findings;
C. Interpret the cumulative previous wet weather monitoring findings;
D. Draw conclusions regarding the cumulative previous wet weather monitoring findings;
E. Provide recommendations for future monitoring activities; and
F. Include an executive summary, introduction, conclusion, and summary of recommendations.

[1] A reporting period is defined as October 1st to September 30th of any year. The first reporting period under this Order is October 1, 2001 to September 30, 2002.

II. Receiving Waters Monitoring Program - - Year Round

Utilizing the findings of the "Previous Monitoring and Future Recommendations Report" discussed above, the Copermittees shall collaborate to develop, submit, conduct, and report on a year round countywide or watershed based Receiving Waters Monitoring Program[2]. The goals of both the countywide and watershed based Receiving Waters Monitoring Program shall be clearly stated. The Receiving Waters Monitoring Program goals shall focus on assessing compliance with this Order, achieving water quality objectives, protecting beneficial uses, and assessing the overall health and long-term water quality trends of receiving waters. For purposes of conducting the countywide or watershed based Receiving Waters Monitoring Program, the Copermittees are encouraged to collaborate with other agencies conducting similar monitoring, such as the Southern California Coastal Water Research Project (SCCWRP), the California Department of Fish and Game, or other municipalities in Southern California. Implementation of the countywide or watershed based Receiving Waters Monitoring Program shall begin within **180 days** of adoption of this Order. The countywide or watershed based Receiving Waters Monitoring Program shall include, at a minimum, the following components:

A. Urban Stream Bioassessment Monitoring
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A. <u>Urban Stream Bioassessment Monitoring</u>

1. The Copermittees shall collaborate to develop and implement an urban stream bioassessment monitoring program. At a minimum, the program shall consist of station identification, sampling, monitoring, and analysis of data for 20 bioassessment stations in order to determine the biological and physical integrity of urban streams within the County of San Diego. In addition to the urban stream bioassessment stations, three reference bioassessment stations shall be identified, sampled, monitored, and analyzed. The selection, sampling, monitoring, and analysis of bioassessment stations shall meet the following requirements:

   a. Each urban stream bioassessment station shall be selected using the following criteria. Each urban stream bioassessment station shall:
      (1) be located within the jurisdiction of a Copermittee; or
      (2) be located within one of the nine watersheds specified in Section J, Table 4 of this Order; and
      (3) be representative of urban stream conditions within one of the nine watersheds specified in Section J, Table 4 of this Order; and
      (4) meet the physical criteria of the California Stream Bioassessment Procedure[3]; and
      (5) to the extent feasible, coincide with the location of an already existing monitoring station used by the California Department of Fish and Game in the conduct of the SDRWQCB's Ambient Bioassessment Program.

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[2] During the first two years, monitoring and reporting will be conducted and reported on a countywide basis. Beginning in the third monitoring period of Order 2001-01, the monitoring and reporting program will shift to a watershed based approach.

[3] California Stream Bioassessment Procedure (Protocol Brief for Biological and Physical/Habitat Assessment in Wadeable Streams), California Department of Fish and Game – Aquatic Bioassessment Laboratory, May 1999.

b. Each bioassessment station shall be monitored twice annually, in May and October of each year, beginning in May 2001. A minimum of three replicate samples shall be collected at each station during each sampling event.

c. Sampling, laboratory, quality assurance, and analysis procedures shall follow the standardized procedures set forth in the California Department of Fish and Game's California Stream Bioassessment Procedure (CSBP). Analysis procedures shall include comparison between station mean values for various biological metrics. Sampling, laboratory, quality assurance, and analytical procedures shall follow the standardized "Non-Point Source Bioassessment Sampling Procedures" for professional bioassessment set forth in the CSBP. In the event that the CSBP "Point-Source Professional Bioassessment Procedure" is performed in place of the "Non Point Source Bioassessment Sampling Procedure," justification and documentation of the procedure shall be submitted with the report. Results of the Urban Stream Bioassessment Monitoring shall be reported annually as part of the overall Receiving Waters Monitoring and Reporting Program for Order No. 2001-01. Reporting of the bioassessment data shall follow the format of the San Diego Regional Water Quality Control Board 1999 Biological Assessment Annual Report[4]. The report shall include:

(1) All physical, chemical and biological data collected in the assessment;
(2) Photographic documentation of assessment and reference stations;
(3) Documentation of quality assurance and control procedures;
(4) Analysis that shall include calculation of the metrics used in both the CSBP and the 1999 Annual Report.
(5) The report shall provide interpretation for comparisons of mean biological and habitat assessment metric values between assessment and reference stations.
(6) Utilize a regional index of biological integrity as part of the analysis.
(7) Electronic data formatted to California Department of Fish and Game Aquatic Bioassessment Laboratory specifications for inclusion in the Statewide Access Bioassessment database.

d. A professional environmental laboratory shall perform all sampling, laboratory, quality assurance, and analytical procedures. While valuable, data collected by volunteer monitoring organizations shall not be submitted in place of professional assessments.

e. Reference stations shall be selected following the recommendations in the 1999 Annual Report, Hughes (1995)[5] and Barbour et. al. (1999)[6]. Reference stations shall be evaluated annually by the Copermittees for suitability and the results included in the annual report. New reference stations will be selected as needed by the Copermittees.

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4 San Diego Regional Water Quality Control Board ,1999 Biological Assessment Annual Report. A Water Quality Inventory Series: Biological and Physical/Habitat Assessment of California Water Bodies. California Department of Fish and Game Office of Spill Prevention and Response, Water Pollution Control Laboratory. December 1999.

5 Hughes, R. M. (1995) Defining Acceptable Biological Status by Comparing with Reference Conditions in Biological Assessment and Criteria: Tools for Water Resource Planning and Decision Making, Wayne S. Davis and Thomas P. Simon eds. Lewis Publishers, Boca Raton, LA.

6 Barbour, M.T. , J Gerritsen, B.D. Synder, and J.B. Stribling (1999) Rapid Bioassessment Protocols For Use in Streams and Wadeable Rivers: Periphyton, Benthic Macroinvertebrates, and Fish. Second Edition. EPA 841-B-99-002.

2. The Copermittees shall design and implement a program to conduct standardized toxicity testing at urban stream bioassessment stations where the bioassessment data indicates significant impairment. When findings indicate the presence of toxicity, a Toxicity Identification Evaluation (TIE) shall be conducted to determine the cause(s) of the toxicity.

B. Long-term Mass Loading Monitoring

For purposes of evaluating long-term trends, the Copermittees shall continue to monitor the five existing long-term mass loading stations as specified in Monitoring and Reporting Program No. 95-76 and amended by Technical Change Order Nos. 1-4. When findings indicate the presence of toxicity, a Toxicity Identification Evaluation (TIE) shall be conducted to determine the cause(s) of the toxicity.

C. Coastal Storm Drain Outfall Monitoring

The Copermittees shall collaborate to develop and implement a monitoring program for discharges of urban runoff from coastal storm drain outfalls. The program shall meet the following requirements:

1. The program shall include rationale and criteria for selection of storm drain outfalls to be monitored.

2. The program shall include collection of samples for analysis of total coliform, fecal coliform, and enterococci, in addition to any other indicators or pathogens identified by the Copermittees.

3. Samples shall be collected at both the storm drain outfall and in the surf zone (at ankle to knee water depths) directly in front of the outfall.

4. Samples shall be collected during both dry and wet weather periods.

5. Exceedances of public health standards for bacteria must be reported to the County Department of Public Health as soon as possible by the Copermittees.

D. Ambient Bay, Lagoon, and Coastal Receiving Water Monitoring

The Copermittees shall collaborate to develop and implement a program to assess the overall health of the receiving water and monitor the impact of urban runoff on ambient receiving water quality. This monitoring shall including San Diego Bay, Mission Bay, Oceanside Harbor, the Pacific Ocean coastline, coastal lagoons and estuaries, and all Clean Water Act section 303(d) water bodies or other environmentally sensitive areas as defined in F.1.b(2)(a)vii of this Order .

E. Toxic Hot Spots Monitoring in San Diego Bay

The Copermittees shall collaborate to develop and implement a program to assess the relative contribution of urban runoff on Toxic Hot Spots in San Diego Bay.

III. Submittal of Receiving Waters Monitoring Program Document

The Principal Permittee shall submit to the SDRWQCB the countywide or watershed based Receiving Waters Monitoring Program within **180 days** of adoption of this Order. The regional or watershed based Receiving Waters Monitoring Program shall describe how the Copermittees will meet the requirements of the components outlined in Section II of this Attachment.

IV. Submittal of Receiving Waters Monitoring Annual Reports

The Principal Permittee shall submit the Receiving Waters Monitoring Annual Report to the SDRWQCB on January 31 of each year, beginning on January 31, 2003.

V. Monitoring Annual Report Requirements

A. Monitoring reports shall provide the data/results, methods of evaluating the data, graphical summaries of the data, and an explanation/discussion of the data for each monitoring program component listed above.

B. Monitoring reports shall include an analysis of the findings of each monitoring program component listed above. The analysis shall identify and prioritize water quality problems. Based on the identification and prioritization of water quality problems, the analysis shall identify potential sources of the problems, and recommend future monitoring and BMP implementation measures for identifying and addressing the sources. The analysis shall also include an evaluation of the effectiveness of existing control measures.

C. Monitoring reports shall include identification and analysis of any long-term trends in storm water or receiving water quality.

D. Monitoring reports shall provide an estimation of total pollutant loads (wet weather loads plus dry weather loads) due to urban runoff for each of the watersheds specified in Section J, Table 4 of Order No. 2001-01.

E. Monitoring reports shall for each monitoring program component listed above, include an assessment of compliance with applicable water quality standards.

F. All monitoring reports shall use a standard report format and shall include the following:

1. A stand alone comprehensive executive summary addressing all sections of the monitoring report;
2. Comprehensive interpretations and conclusions; and
3. Recommendations for future actions.

G. All monitoring reports submitted to the Principal Permittee or the SDRWQCB shall contain the certified perjury statement described in Standard Reporting Requirements in Attachment C section B.10.d.

H. All monitoring reports shall be reviewed prior to submittal to the SDRWQCB by a committee (consisting of no less than three members). All review comments shall also be submitted to the SDRWQCB.

I. All monitoring reports shall be submitted in both electronic and paper formats.

J. All monitoring reports shall describe monitoring station locations by latitude and longitude coordinates, frequency of sampling, quality assurance/quality control procedures and sampling and analysis protocols.

K. Monitoring programs and reports shall comply with Section VI of Attachment B, as well as Attachment C.

VI. Standard Monitoring Requirements

A. All monitoring activities shall meet the following requirements:

1. Monitoring and Records [40 CFR 122.41(j)(1)]

Samples and measurements taken for the purpose of monitoring shall be representative of the monitored activity.

2. Monitoring and Records [40 CFR 122.41(j)(2)] [California Water Code § 13383(a)]

The discharger shall retain records of all monitoring information, including all calibration and maintenance records and all original strip chart recordings for continuous monitoring instrumentation, copies of all reports required by this Order, and records of all data used to complete the application for this permit, for a period of at least three years from the date of the sample, measurement, report or application. This period may be extended by request of the SDRWQCB at any time.

3. Monitoring and Records [40 CFR 122.21(j)(3)]

Records of monitoring information shall include the information requested in Attachment B and the following:

a. The date, exact place, and time of sampling or measurements;
b. The individual(s) who performed the sampling or measurements;
c. The date(s) analyses were performed;
d. The individual(s) who performed the analyses;
e. The analytical techniques or methods used; and
f. The results of such analyses.

4. Monitoring and Records [40 CFR 122.21(j)(4)]

Monitoring results must be conducted according to test procedures approved under 40 CFR part 136 unless other test procedures have been specified in this Order.

5. Monitoring and Records [40 CFR 122.21(j)(5)]

The Clean Water Act provides that any person who falsifies, tampers with, or knowingly renders inaccurate any monitoring device or method required to be maintained under this Order shall, upon conviction, be punished by a fine of not more than $10,000, or by imprisonment for not more than two years, or both. If a conviction of a person is for a violation committed after a first conviction of such person under this paragraph, punishment is a fine of not more than $20,000 per day of violation, or by imprisonment of not more than four years, or both.

6. Monitoring and Records [40 CFR 122.41(k)(2)]

The Clean Water Act provides that any person who knowingly makes any false statement, representation, or certification in any record or other document submitted or required to be maintained under this permit, including monitoring reports or reports of compliance or non-compliance shall, upon conviction, be punished by a fine of not more than $10,000 per violation, or by imprisonment for not more than six months per violation, or by both.

7. Monitoring Reports [40 CFR 122.41(l)(4)

Monitoring results shall be reported at the intervals specified elsewhere in this Order.

8. Monitoring Reports [40 CFR 122.41(l)(4)(ii)]

If the discharger monitors any pollutant more frequently than required by the permit using test procedures approved under 40 CFR part 136, unless otherwise specified in the Order, the results of this monitoring shall be included in the calculation and reporting of the data submitted in the reports requested by the SDRWQCB.

9. Monitoring Reports [40 CFR 122.41(l)(4)(iii)]

Calculations for all limitations which require averaging of measurements shall utilize an arithmetic mean unless otherwise specified by the SDRWQCB in the Order.

U.S. Department of the Interior
U.S. Geological Survey

Prepared in cooperation with the
FEDERAL HIGHWAY ADMINISTRATION

A Synopsis of Technical Issues for Monitoring Sediment in Highway and Urban Runoff

Open-File Report 00-497

A Contribution to the
NATIONAL HIGHWAY RUNOFF DATA AND METHODOLOGY SYNTHESIS

U.S. Department
of Transportation

USGS
science for a changing world

R0022751

U.S. Department of the Interior
U.S. Geological Survey

A Synopsis of Technical Issues for Monitoring Sediment in Highway and Urban Runoff

By GARDNER C. BENT, JOHN R. GRAY, KIRK P. SMITH, and
G. DOUGLAS GLYSSON

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A Contribution to the
NATIONAL HIGHWAY RUNOFF DATA AND METHODOLOGY SYNTHESIS

Prepared in cooperation with the
FEDERAL HIGHWAY ADMINISTRATION

Northborough, Massachusetts
2001

R0022752

U.S. DEPARTMENT OF THE INTERIOR
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U.S. GEOLOGICAL SURVEY
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R0022753

PREFACE

Knowledge of the characteristics of highway runoff (concentrations and loads of constituents and the physical and chemical processes which produce this runoff) is important for decision makers, planners, and highway engineers to assess and mitigate possible adverse impacts of highway runoff on the Nation's receiving waters. In October 1996, the Federal Highway Administration and the U.S. Geological Survey began the National Highway Runoff Data and Methodology Synthesis to provide a catalog of the pertinent information available; to define the necessary documentation to determine if data are valid (useful for intended purposes), current, and technically supportable; and to evaluate available sources in terms of current and foreseeable information needs. This paper is one contribution to the National Highway Runoff Data and Methodology Synthesis and is being made available as a U.S. Geological Survey Open-File Report pending its inclusion in a volume or series to be published by the Federal Highway Administration. More information about this project is available on the World Wide Web at http://ma.water.usgs.gov/fhwa/runwater.htm

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yards	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3

NOTE: Volumes greater than 1000 l shall be shown in m^3.

Symbol	When You Know	Multiply By	To Find	Symbol
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5(F-32)/9 or (F-32)/1.8	Celcius temperature	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.71	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3

Symbol	When You Know	Multiply By	To Find	Symbol
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

(Revised September 1993)

* SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

R0022757

A Synopsis of Technical Issues for Monitoring Sediment in Highway and Urban Runoff

By Gardner C. Bent, John R. Gray, Kirk P. Smith, *and* G. Douglas Glysson

Abstract

Accurate and representative sediment data are critical for assessing the potential effects of highway and urban runoff on receiving waters. The U.S. Environmental Protection Agency identified sediment as the most widespread pollutant in the Nation's rivers and streams, affecting aquatic habitat, drinking water treatment processes, and recreational uses of rivers, lakes, and estuaries. Representative sediment data are also necessary for quantifying and interpreting concentrations, loads, and effects of trace elements and organic constituents associated with highway and urban runoff. Many technical issues associated with the collecting, processing, and analyzing of samples must be addressed to produce valid (useful for intended purposes), current, complete, and technically defensible data for local, regional, and national information needs. All aspects of sediment data-collection programs need to be evaluated, and adequate quality-control data must be collected and documented so that the comparability and representativeness of data obtained for highway- and urban-runoff studies may be assessed.

Collection of representative samples for the measurement of sediment in highway and urban runoff involves a number of interrelated issues. Temporal and spatial variability in runoff result from a combination of factors, including volume and intensity of precipitation, rate of snowmelt, and features of the drainage basin such as area, slope, infiltration capacity, channel roughness, and storage characteristics. In small drainage basins such as those found in many highway and urban settings, automatic samplers are often the most suitable method for collecting samples of runoff for a variety of reasons. Indirect sediment-measurement methods are also useful as supplementary and (or) surrogate means for monitoring sediment in runoff. All of these methods have limitations in addition to benefits, which must be identified and quantified to produce representative data. Methods for processing raw sediment samples (including homogenization and subsampling) for subsequent analysis for total suspended solids or suspended-sediment concentration often increase variance and may introduce bias. Processing artifacts can be substantial if the methods used are not appropriate for the concentrations and particle-size distributions present in the samples collected.

Analytical methods for determining sediment concentrations include the suspended-sediment concentration and the total suspended solids methods. Although the terms suspended-sediment concentration and total suspended solids are often used interchangeably to describe the total concentration of suspended solid-phase material, the analytical methods differ and can produce substantially different results. The total suspended solids method, which commonly is used to produce highway- and urban-runoff sediment data, may not be valid for studies of runoff water quality. Studies of fluvial and highway-runoff sediment data indicate that analyses of samples by the total suspended solids method tends to underrepresent the true sediment concentration, and that relations between total suspended solids and suspended-sediment concentration are not transferable from

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site to site even when grain-size distribution information is available. Total suspended solids data used to calculate suspended-sediment loads in highways and urban runoff may be fundamentally unreliable. Consequently, use of total suspended solids data may have adverse consequences for the assessment, design, and maintenance of sediment-removal best management practices. Therefore, it may be necessary to analyze water samples using the suspended-sediment concentration method.

Data quality, comparability, and utility are important considerations in collection, processing, and analysis of sediment samples and interpretation of sediment data for highway- and urban-runoff studies. Results from sediment studies must be comparable and readily transferable to be useful to resource managers and regulators. To meet these objectives, supporting ancillary information must be available to document the methods and procedures that are used and to describe quality-assurance and quality-control procedures that are used in the studies. Valid, current, and technically defensible protocols for collecting, processing, and analyzing sediment data for the determination of water quality in highway and urban runoff therefore need to be documented with study results.

INTRODUCTION

Recognition of the importance of sediment as a water-quality constituent has increased dramatically in recent years. The U.S. Environmental Protection Agency (2000) identified sediment as the most widespread pollutant in the Nation's rivers and streams, in that sediment affects aquatic habitat, drinking-water treatment processes, and recreational uses of rivers, lakes, and estuaries. To address the combined, cumulative impacts of both point and nonpoint sources of sediment, the U.S. Environmental Protection Agency (USEPA) has adopted a watershed approach, of which total maximum daily loads (TMDLs) are a part (U.S. Environmental Protection Agency, 1998 and 1999).

In addition to sediment itself being a major pollutant, many trace elements, such as copper, zinc, cadmium, chromium, lead, and nickel—constituents often detected in highway runoff—are associated with sediments (Gupta and others, 1981; Horowitz, 1995). Some organic constituents associated with highway runoff are also associated with sediments (Lopes and Dionne, 1998). Sediment in highway runoff is a potential problem as a physical contaminant and as a source of potentially toxic substances to the local ecosystem (Schueler, 1997; Buckler and Granato, 1999). Hence, sediment in highway runoff can be a dominant factor in water quality, particularly when selected trace elements or organic constituents are associated with the sediment.

Highways affect sediment transport in runoff through several processes. Reduced infiltration from impervious surfaces, rapid concentration of flow with minimal flow resistance, and relatively high slopes of roadside drainage structures combine to increase velocities, volumes, and peaks of storm runoff, thus increasing the potential for erosion and increased entrainment of sediment. Materials entrained in highway runoff from road-surface and vehicle degradation can be discharged to receiving streams, as can materials such as sand or cinders that may be applied for traction on snow or ice. Particles from atmospheric deposition that include combustion and other by-products from vehicles also can be entrained in highway runoff.

This report addresses technical issues pertinent to the methods for the collection, processing, and analysis of sediment samples to determine the concentration and physical characteristics of sediment in highway and urban runoff, best management practice (BMP) structures, and receiving waters. Data-quality issues and appropriate quality-assurance techniques for sediment data collection and laboratory-analysis methods are also discussed. Although this report focuses on sediment-transport issues related to highway use, information presented in this report also is applicable to many issues related to sediment in urban runoff. Sediment-transport issues related to highway construction are presented within the Nonpoint Pollution Discharge Elimination System (NPDES) (U.S. Environmental Protection Agency, 1992; 1995) and are not included herein. Many of the techniques discussed herein may be used to monitor receiving waters, although those discussed by Edwards and Glysson (1999) may be more appropriate in fluvial

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systems. Hence, this paper focuses on valid, current, and technically defensible protocols for collecting, processing, and analyzing sediment data for the determination of water quality in highway and urban runoff.

SEDIMENT CONCEPTS RELATED TO HIGHWAY RUNOFF

Sediment comprises particles derived from rocks, biological materials, or chemical precipitates that are transported by, suspended in, or deposited in flowing water (American Society for Testing and Materials, 1997b). Highway sediments can be a mix of materials including pavement dust and particles; atmospheric dust, natural soils, traction sand and cinders; vehicle rust particles; tire dust and particles; trash; and plant and leaf material. The mode of transport can be described by the origin of the material as bed-material load and wash load; or operationally (as measured by sediment samplers) as suspended load and bedload (International Standards Organization, written commun., 2000). Wash load is material atypical of the bottom-material size distribution that tends to flow through a reach without significant interaction with the bed. Suspended-sediment load is material carried in suspension by turbulence. Suspended particles less than about 0.04 mm are typically well mixed within the water column profile (Butler and others, 1996a). As particle-size distribution (PSD) increases to include sand-size material (larger than 0.062 mm median diameter), a vertical gradient may form, with largest particles concentrating near the bed. For example, the theoretical vertical distribution of sediments in the water column calculated using the Rouse equation (Graf, 1996) for highway sediments of different grain sizes are presented in figure 1. The concentration distribution (relative to a normalized sampling point that is at 0.1 of the water level above the bed) is uniform for finer particles (diameters less than 0.062 mm). As grain size increases from very fine sand to coarse sand (0.062 to 1.0 mm), however, the relative concentration increases with depth as a function of increasing grain size under standard conditions (fig. 1). These theoretical concentration distributions compare favorably with patterns in data that have been collected in fluvial systems (Guy, 1970) and in the results of an experiment designed to assess the vertical distribution of sediments in a small highway drainage pipe (Smith, 2000).

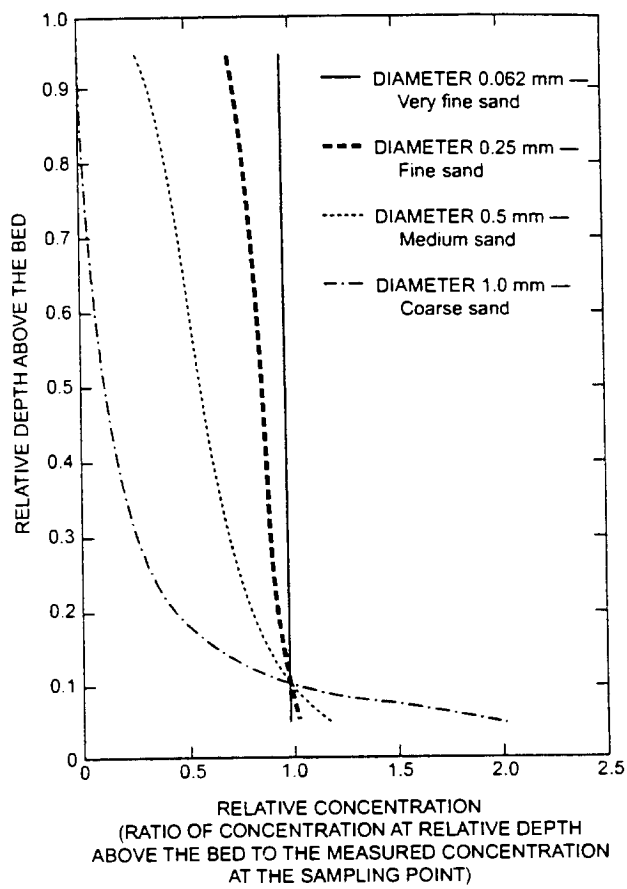

Figure 1. Theoretical vertical distribution of sediments (specific gravity 2.65) in a runoff drainage pipe with a 5-percent slope in open channel flow under standard conditions based on the Rouse equation (Graf, 1996).

Bedload is material that moves by rolling, sliding, or saltating along the channel bottom. Butler and others (1996a) indicate that bedload particle sizes are typically larger than about 0.3 mm in storm sewers. This distinction, however, is not quantitative because it depends on several hydraulic variables including channel slope, specific gravity of solids, particle shape, and flow energy. Material composing the bed at a low flow may move as bedload at a higher flow, and as suspended load at still higher flows. In comparison, bedload in fluvial systems rarely includes sediment that is finer than 0.1–0.2 mm in diameter, because once disturbed, the finer particle sizes go directly into suspension (Gomez and others, 1991). Because storm flows can vary from zero to peak flow in minutes, the

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dominant phase of transport can change rapidly. For example, 0.25 mm material might be transported predominately as bedload at flows of 1 ft^3/s, but at flows of 10 ft^3/s the dominant phase of transport might be suspended sediment.

Sediment transport in highway and urban runoff is controlled by precipitation runoff and the availability of erodible and (or) transportable sediments. The amount and timing of runoff is largely dependent on rainfall intensity and depth. Rainfall intensity has a two-fold effect on entrainment and transport of sediment through (1) raindrop splash erosion and (2) through sheet flow. The kinetic energy imparted by rain, which causes splash erosion when incident on sediment, increases exponentially with rainfall intensity (Hudson, 1981). The volume and velocity of sheet flow tends to increase with rainfall volume and intensity, entraining sediment from paved surfaces. Large flows resulting from high-intensity rains can lead to suspension and transport of sediment on paved areas and in drainage structures. Therefore, a lone sample collected during the first 30 minutes of the runoff period, such as is required by the U.S. Environmental Protection Agency (Bailey, 1993; Stillwell and Bailey, 1993), is a somewhat arbitrary requirement and may not adequately describe sediment concentrations associated with the "first flush" of runoff sediment. Also, because of the effects of varying rainfall intensities on the timing and the magnitude of runoff, a maximum sampling duration of 3 hours, as recommended by the USEPA (Bailey, 1993; Stillwell and Bailey, 1993), may result in substantial underestimation of sediment discharges for extended runoff periods.

The erosive capacity of runoff from highways and urban areas can be substantial because runoff from paved areas, ditches, and storm drains can be hydraulically supercritical and turbulent. The area contributing to surface runoff is usually small, water-surface slope is commonly relatively steep, and surface roughness is usually low. Under these conditions, runoff quickly becomes concentrated. Although some coarser sediment can move as bedload, most highway drainage systems are designed to maintain sediments suspended in runoff so that the volumetric capacities of the highway conveyance structures are not diminished (Butler and others, 1996a).

Most highway runoff sediment-monitoring programs are implemented in areas ranging from a fraction of an acre to several square miles. These are small areas compared to the median drainage area of 296 mi^2 for the 1,593 U.S. Geological Survey (USGS) gaging stations listed in the USGS historical daily-value suspended-sediment and ancillary data base (U.S. Geological Survey, 1999b). Monitoring sediment and flow in small drainage systems in which runoff responds rapidly to rainfall usually requires a combination of manual and automatic methods for data collection (Robertson and Roerish, 1999). Water-discharge and sediment-concentration data are required to calculate suspended-sediment transport (Porterfield, 1972; Koltun and others, 1994). Samples for sediment-concentration analyses should be collected during the rising limb, peak, and falling limb of the runoff hydrograph to describe adequately variations in sediment concentrations for the runoff period. This data-collection scheme is important because the relation between the concentration of suspended material and runoff is generally not the same on the rising limb of the hydrograph as the falling limb of the hydrograph (fig. 2) (Porterfield, 1972; Mustard and others, 1987). In the case of highway runoff, consecutive samples may need to be collected only minutes apart, particularly on the rising limb of the hydrograph, due to large temporal variations in sediment concentrations, and a "first-flush" effect that may occur at highway and urban monitoring sites where sediments have accumulated between runoff periods. The "first-flush" effect with sediment in highway runoff results when a period of the rising limb of the runoff hydrograph is disproportionally enriched in sediment compared to the remaining period of the hydrograph (fig. 2), and has been quantified in highway-runoff studies by Patrick (1975), Ellis (1976), Ellis and others (1981), and Mustard and others (1987). The "first-flush" effect is likely due to an accumulation of fine sediments that are entrained in the initial runoff. These finer sediments available for transport could have been previously deposited in the drainage conveyance structures or could have accumulated on the road surface, usually near the curb or road edge, which is generally the flow path of runoff going to the

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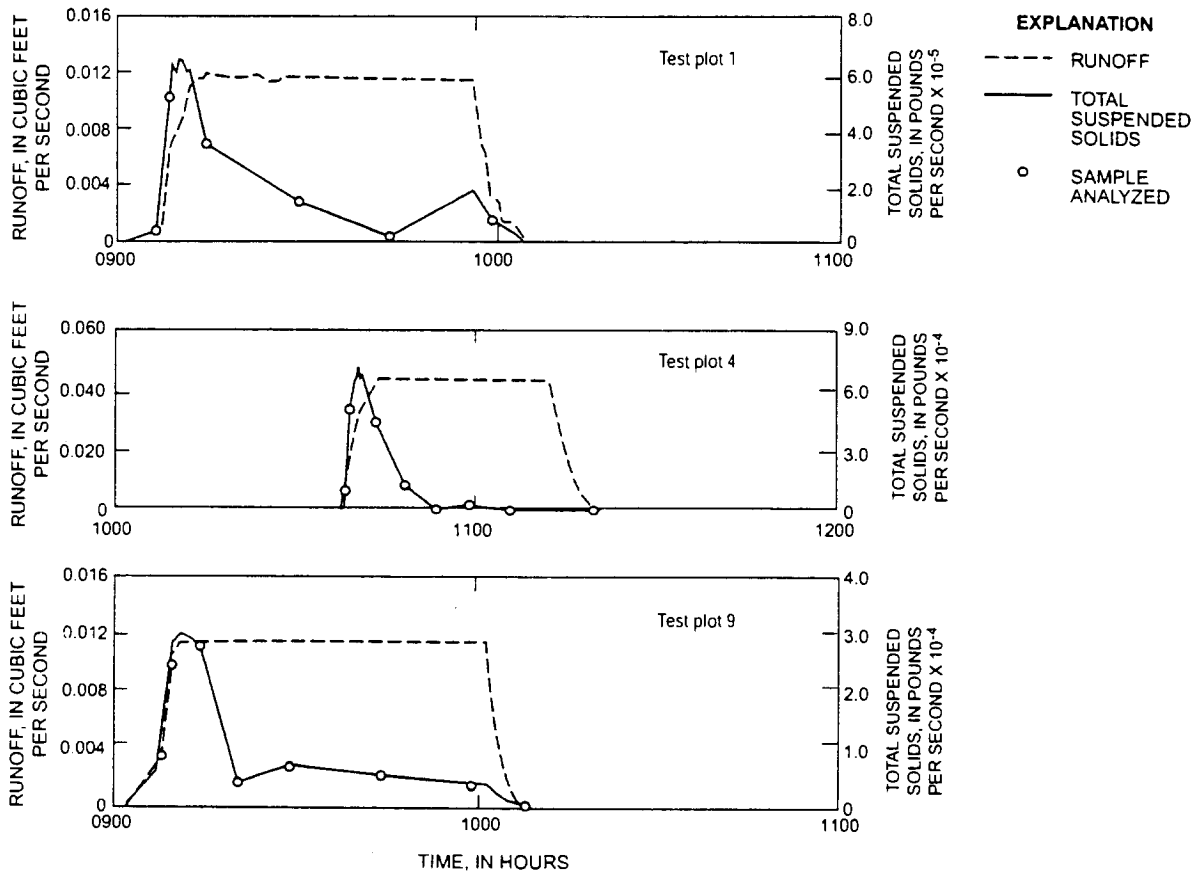

Figure 2. Runoff and washoff load of total suspended solids from test plots in Lakewood, Colorado, June 3, 1980 (modified from Mustard and others, 1987).

drainage conveyance structures from, or since, the last runoff period. For example, Gupta and others (1981) reported that 85 to 90 percent of street debris (solids) was within 12 in. of the curb. However, sediment transport at many highway and urban sites may vary with precipitation intensity and therefore the "first flush" may not represent the maximum sediment concentration or load that occurs during a runoff period.

Characteristics important to the monitoring, analysis, interpretation, and ultimately the treatment of water quality in highway and urban runoff include sediment concentrations and PSDs. Review of studies from the United States, Australia, Canada, France, Sweden, and the United Kingdom related to sediment runoff from highways, streets, and urban areas over the last 25 years (table 1) have shown concentrations of total suspended solids (TSS) ranging from 4 to 129,000 mg/L (table 2). In comparison, mean TSS concentrations in table 2 range from 29 to about 18,000 mg/L. The median particle size (d_{50}) of sediments collected in these studies range from 0.013 to 1.00 mm (table 3). These particle sizes range from about medium silt to very coarse sand (Guy, 1969; Folk, 1980), with sand-size particles being those larger than 0.062 mm (table 4).

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Table 1. List of selected studies of sediment in highway and urban runoff

[**Report type:** D/I, data and interpretation; S, summary. **Location:** I, Interstate; SR, State Route. >, greater than; %, percent]

Reference	Report type	Location	Site type	Study year
		United States		
Asplund and others, 1982	D/I	Seattle, Wash. (I-5)	urban highway	1979–80
	D/I	Seattle, Wash. (I-5 with grit)	urban highway	1979–80
	D/I	Montlake, Wash. (SR-520)	suburban highway	1979–80
	D/I	Vancouver, Wash.	highway	1979–80
	D/I	Snoqualmie Pass, Wash.	agricultural highway	1979–80
	D/I	Montesano, Wash.	agricultural highway	1979–80
	D/I	Pasco, Wash.	urban highway	1979–80
	D/I	Spokane, Wash.	agricultural highway	1979–80
	D/I	Pullman, Wash. (site 9)	agricultural highway	1979–80
Mustard and others, 1987	D/I	Lakewood, Colo. (4-lane street)	city street	1980
Smith and Lord, 1990	S	Selected Highways	highway	1976–77
Driscoll and others, 1991	S	Selected Urban Highways	urban highway	1980–90
	S	Selected Rural Highways	rural highway	1980–90
	S	Selected Urban Highways	urban highway	1980–90
	S	Selected Rural Highways	rural highway	1980–90
Moser, 1996	D/I	Silverthorne, Colo. (I-70)	highway	1994
	D/I	Silverthorne, Colo. (I-70)	highway	1994
	D/I	Silverthorne, Colo. (I-70)	highway	1994
Sansalone and Buchberger, 1996	D/I	Cincinnati, Ohio (I-75)	urban highway	1995
Sansalone and others, 1996	D/I	Cincinnati, Ohio (I-75)	urban highway	1995
Corsi and others, 1997	D/I	Southeastern Wisconsin	>90% urban land use	1975–96
Dudley and others, 1997	D/I	New Sharon, Maine (SR-2)	rural highway	1992–93
	D/I	New Sharon, Maine (SR-27)	rural highway	1992–93
Sansalone and others, 1998	D/I	Cincinnati, Ohio (I-75)	urban highway	1995–97
Waschbusch and others, 1999	D/I	Madison, Wisc.	residential streets	1994–95
		Austrialia		
Ball and Abustan, 1995	D/I	Sydney	residential area	1994
		Canada		
Vermette and others, 1987	D/I	Hamilton, Ontario	street	Not Reported
		France		
Roger and others, 1998	D/I	Herault Region	highway	1993–94
Andral and others, 1999	D/I	France	highway	1993–94
Legret and Pagotto, 1999	D/I	Loire-Atlantique	rural highway	1995–96
		Sweden		
Viklander, 1998	D/I	Lulea	street	1996
		United Kingdom		
Ellis and others, 1981	D/I	London, England	residential and others	Not Reported
Pratt and Adams, 1981	D/I	Nottingham, England	residential streets	1979–80
Ellis and Harrop, 1984	D/I	London, England	highway	Not Reported
Ellis and others, 1987	D/I	London, England	highway	Not Reported
Butler and others, 1992	D/I	Lambeth, London, England	urban highway	Not Reported
Boxall and Maltby, 1995	D/I	United Kingdom	urban highway	Not Reported

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Table 1. List of selected studies of sediment in highway and urban runoff—*Continued*

Reference	Report type	Location	Site type	Study year
		United Kingdom—*Continued*		
Butler and others, 1996a	S	United Kingdom	Not Reported	Not Reported
	S	United Kingdom	Not Reported	Not Reported
		No Specified Location		
Bertrand-Krajewski and others, 1993	S	Not Reported	Not Reported	Not Reported

Table 2. Sediment concentrations measured in highway and urban runoff

[n: number of sediment concentration analyses (data points); g/m^3, grams per cubic meter; mg/L, milligrams per liter; --, no data]

Reference	Sediment concentrations reported (mg/L)			Comments
	n	Range	Mean	
Asplund and others, 1982	54	32–848	--	Urban highway
	9	50–1,370	--	Urban highway
	43	76–894	--	Suburban highway
	61	13–168	--	Highway
	12	23–586	--	Agricultural highway
	27	51–1,260	--	Agricultural highway
	17	19–587	--	Urban highway
	6	67–2,490	--	Agricultural highway
	6	14–522	--	Agricultural highway
Mustard and others, 1987	9	27–150	83	City street
Smith and Lord, 1990	159	4–1,156	261	Highway
Driscoll and others, 1991	16	51–406	--	Rainfall, urban highway
	8	9–126	--	Rainfall, rural highway
	9	61–752	--	Snowmelt, urban highway
	6	11–465	--	Snowmelt, rural highway
Moser, 1996	30	12–854	213	Rainfall, highway
	9	1,948–69,141	18,036	Snowmelt, highway
Sansalone and Buchberger, 1996	2	84–127	--	Rainfall, urban highway
Sansalone and others, 1996	8	510–3,200	1,419	Snowmelt, urban highway
Corsi and others, 1997	--	17–297	139	>90% urban land use
Dudley and others, 1997	35	18–129,000	--	Rural highway
	27	92–114,000	--	Rural highway
Sansalone and others, 1998	13	29–259	131	Rainfall, urban highway
Waschbusch and others, 1999	--	--	67–99	Residential streets
Andral and others, 1999	8	15–58	29	Highway
Legret and Pagotto, 1999	49	16–267	71	Rural highway
Ellis and others, 1987	34	--	156 g/m^3 and 194 g/m^3	Highway
Butler and others, 1996a	--	50–1,000	--	Stormwater solids
		10–200	--	Grit
Bertrand-Krajewski and others, 1993	--	21–2,582	--	Roads, curbs, runoff, and sewers

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Table 3. Particle-size distribution measured in highway and urban runoff

[n: number of particle-size analyses (data points). **Median (d_{50}):** d_{50} median diameter of particles. **Comments:** Modal is the value of the most commonly occurring particle size. phi, log2 of the particle diameter; µm, micrometers; %, percent; >, actual value is greater than value shown; <, actual value is less than value shown; --, no data]

Reference	Particle-size distribution				Comments
	n	Median (d_{50})	Mean	Range	
Sansalone and others, 1998	13	555 µm	570 µm	370–875 µm	d_{50}
	13	--	--	1–10%	>62 µm
Waschbusch and others, 1999	--	--	--	75%	>250 µm
		--	--	85%	>62 µm
		--	--	5%	<63 µm
Ball and Abustan, 1995	--	40-60 µm	--	--	--
Vermette and others, 1987	8	--	--	354–707 µm	modal size class
	8	--	2.1 phi	1.7–2.9 phi	--
Roger and others, 1998	--	--	--	86%	<50 µm
		--	--	53%	500–1,000 µm
Andral and others, 1999	8	86%	86%	82–91%	<50 µm
	8	13 µm	--	10–16 µm	d_{50}
Viklander, 1998	--	--	--	1,000–3,000 µm	d_{50}
Ellis and others, 1981	--	--	--	2 and 20 µm	bimodal
Pratt and Adams, 1981	1	500 µm	--	--	--
Ellis and Harrop, 1984	2	--	--	650–1,400 µm	--
Butler and others, 1996a	--	60 µm	--	20–100 µm	d_{50}
		750 µm	--	300–1,000 µm	d_{50}
Bertrand-Krajewski and others, 1993	--	30–1,000 µm	--	--	--

Table 4. Recommended particle-size classes for sediment analysis

[Modified from Guy, 1969. **Phi value:** Maximum size of the given class. --, not expressed in terms of micrometers; NA, not applicable]

Class name	Metric units		Phi value
	Millimeters	Micrometers	
Boulders	>256	--	NA
Large cobbles	256–128	--	-8
Small cobbles	128–64	--	-7
Very coarse gravel	64–32	--	-6
Coarse gravel	32–16	--	-5
Medium gravel	16–8.0	--	-4
Fine gravel	8.0–4.0	--	-3
Very fine gravel	4.0–2.0	--	-2
Very coarse sand	2.0–1.0	2,000–1,000	-1
Coarse sand	1.0–0.50	1,000–500	0
Medium sand	0.50–0.25	500–250	+1
Fine sand	0.25–0.125	250–125	+2
Very fine sand	0.125–0.062	125–62	+3
Coarse silt	0.062–0.031	62–31	+4
Medium silt	0.031–0.016	31–16	+5
Fine silt	0.016–0.008	16–8	+6
Very fine silt	0.008–0.004	8–4	+7
Coarse clay	0.004–0.0020	4–2	+8
Medium clay	0.0020–0.0010	2–1	+9
Fine clay	0.0010–0.0005	1–0.5	+10
Very fine clay	0.0005–0.00024	0.5–0.24	+11

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SAMPLE-COLLECTION METHODS

The collection of representative samples for measurement of sediment in highway and urban runoff involves a number of interrelated issues. The temporal and spatial variability in runoff can be large because of a combination of factors including volume and intensity of precipitation, rate of snowmelt, and features of the drainage basin such as drainage area, slope, infiltration capacity, channel roughness, and storage characteristics. As the runoff rate increases, the stage (water level) and (or) mean velocity increases also. Rapid changes in flow may be associated with rapid changes in the sediment concentration, PSD, and density distribution. For example, Butler and others (1996a, 1996b) indicate that sediments accumulated in pipes may be mobilized (as bedload or suspended load) or remain immobile depending on concentration and size distribution of the sediment, and the energy of flow. Therefore, measurement of precipitation and flow are necessary for measurement and interpretation of sediment transport in highway and urban runoff systems (Church and others, 1999). Information necessary for measurement and interpretation of precipitation and runoff flows in highway and urban systems is discussed by Church and others (1999) and so is not included herein. The complexity of the precipitation-runoff-transport process necessitates sampling plans and methods that characterize the temporal and spatial variability in sediment transport in these systems.

Sampling plans for the study of nonpoint-source contamination may include discrete and (or) composite sampling by manual and (or) automatic sampling methods (U.S. Environmental Protection Agency, 1992). Discrete samples (also referred to as grab or dip samples) may represent sediment concentrations for only a short period of time. Composite samples are mixed or combined samples that should be flow-weighted to represent concentrations and loads during the monitoring period (U.S. Environmental Protection Agency, 1992; Gray and Fisk, 1992). Discrete samples collected during a runoff period may be physically composited and analyzed as one sample or mathematically composited from an analysis of multiple discrete samples (Driscoll and others, 1991). Discrete and (or) composite samples may be spatially representative—such as those collected by the Equal Discharge Increment (EDI) or Equal Width Increment (EWI) methods (Edwards and Glysson, 1999)—or may be representative of only the point in the stream from which the sample was collected. Automatic sampling methods include pumping samplers as well as passive devices that are designed to collect a discrete sample. The U.S. Environmental Protection Agency (1992) describes some of the relative advantages and disadvantages of manual and automatic sampling techniques (table 5). The remoteness or inaccessibility of some study sites makes it difficult to monitor runoff periods manually, and it can be difficult to get personnel to the sites before the onset of runoff. Costs associated with deployment of trained and properly equipped personnel, in addition to uncertainties related to the location and timing of runoff, can be prohibitive for manual sampling of storm-runoff periods. For example, Thiem and others (1998) employed a meteorologist to predict storm events and still had difficulty in implementing manual sampling efforts because storm runoff was predicted with an accuracy of about 50 percent. The difficulty in collecting a relatively large number of samples during storm runoff and the dangers to field personnel operating in adverse conditions (including traffic, weather, reduced visibility, and rapid changes in discharge) reduces the practicality of manual sampling efforts. In contrast, automatic samplers can be deployed before, and samples can be retrieved after cessation of storm runoff, thereby reducing logistics and increasing the safety for field personnel (Federal Interagency Sedimentation Project, 1981; U.S. Environmental Protection Agency, 1992). Also, the large temporal and spatial uncertainty in precipitation and runoff, and the coordination possible between automatic precipitation, flow, and water-quality-measurement instruments and automatic samplers (Church and others, 1996) favor the use of these devices in the monitoring of runoff quality in highway and urban systems. For example, Lewis (1996) describes a means for activating an automatic sediment sampler based on real-time turbidity measurements. Edwards and Glysson (1999) describe manual methods for sediment sample collection, which are typically more suitable for monitoring receiving waters than highway- and urban-drainage systems. The automatic samplers commonly used to sample highway- and urban-drainage systems are discussed as follows.

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Table 5. Comparison of manual and automatic sampling techniques

[Modified from U.S. Environmental Protection Agency, 1992.]

Sample method	Advantages	Disadvantages
Manual grabs	Appropriate for all pollutants Minimum equipment required	Labor-intensive Environment possibly dangerous to field personnel May be difficult to get personnel and equipment to the storm water outfall within the 30-minute requirement Possible human error
Manual flow-weighted composites (multiple grabs)	Appropriate for all pollutants Minimum equipment required	Labor-intensive Environment possibly dangerous to field personnel Human error may have significant impact on sample representativeness Requires flow measurements taken during sampling
Automatic grabs	Minimizes labor requirements Low risk of human error Reduced personnel exposure to unsafe conditions Sampling may be triggered remotely or initiated according to present conditions	Samples collected for oil and grease may not be representative Automatic samplers can not properly collect samples for volatile organic compounds analysis Costly if numerous sampling sites require the purchase of equipment Requires equipment installation and maintenance Requires operator training May not be appropriate for pH and temperature May not be appropriate for parameters with short holding times (for example, fecal streptococcus, fecal coliform, chlorine) Cross-contamination of aliquot if tubing/bottles not washed
Automatic flow-weighted composites	Minimizes labor requirements Low risk of human error Reduced personnel exposure to unsafe conditions May eliminate the need for manual compositing of aliquots Sampling may be triggered remotely or initiated according to on-site conditions	Not acceptable for volatile organic compounds sampling Costly if numerous sampling sites require the purchase of equipment Requires equipment installation and maintenance, may malfunction Requires initial operator training Requires accurate flow-measurement equipment tied to sampler Cross-contamination of aliquot if tubing/bottles not washed

Automatic Samplers

Automatic samplers include active (pumping samplers) and passive sampling devices. Automatic pumping samplers typically collect water from the water column by suction and control the sampling rate using the pump speed (Dick, 1996). Passive sampling devices typically are installed in the flow path and control the sampling rate by placement, orientation, and design of the water intake. Each type of sampler has benefits and design limitations, which must be recognized and quantified to produce representative data.

Pumping Samplers

Automatic pumping-type samplers (fig. 3) generally consist of (1) a pump to draw suspended-sediment samples from the water column and, in some cases, to provide a back flush to clear the sampler intake before

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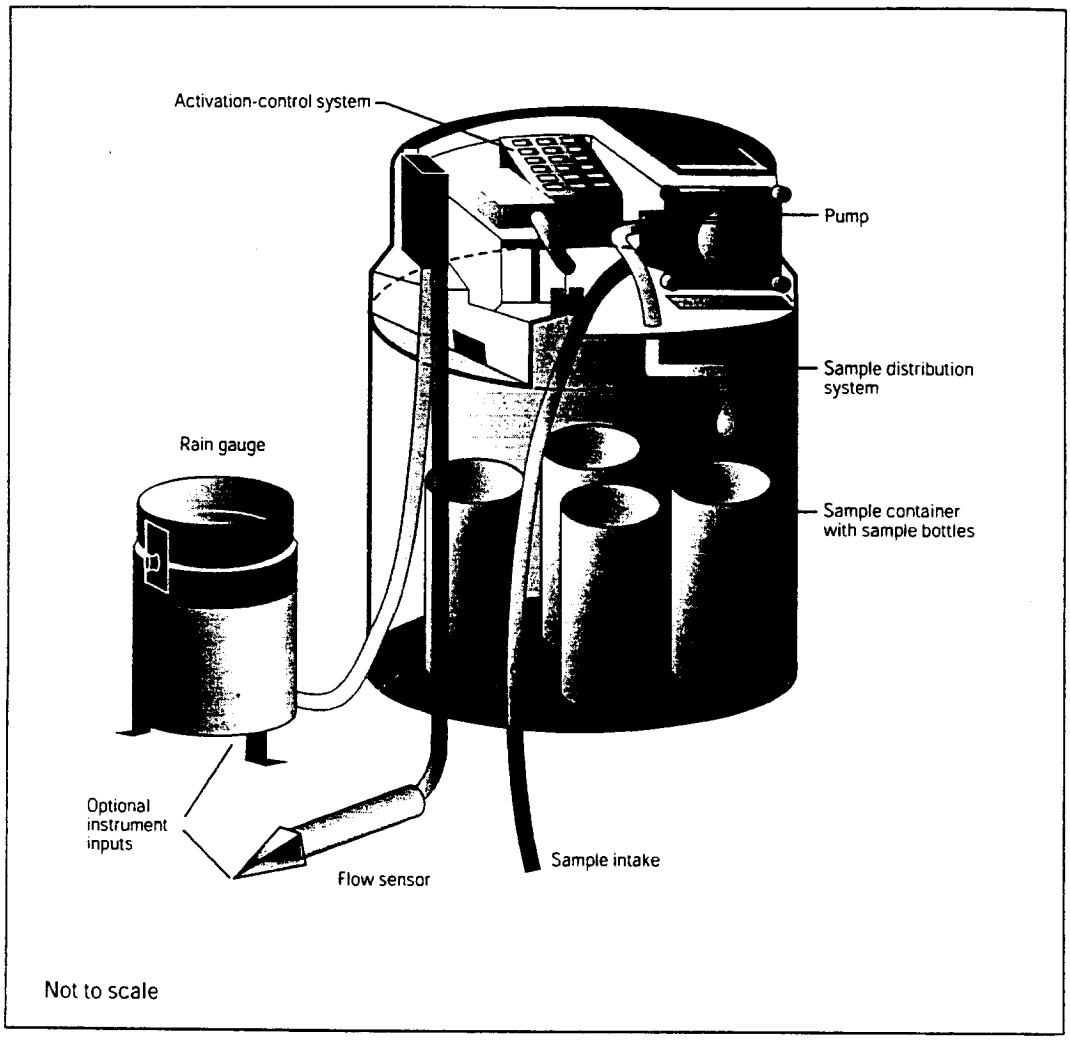

Figure 3. Automatic pumping sampler (modified from U.S. Environmental Protection Agency, 1992).

or after each sampling cycle; (2) a sample-container unit to hold sample bottles in position for filling; (3) a sample distribution system to divert a pumped sample to one or more sample collection bottle(s); (4) an activation-control system that activates the sampling cycle on a time interval, stage, rate-of-stage-change, or from an external signal (such as in response to a telephone call or a signal generated by a data logger); and (5) an intake system through which samples are drawn from a point in the water column's cross-section. Ideally, this combination of components should be designed to meet the following criteria based on site-specific conditions:

- A suspended-sediment sample should be delivered from the water column to the sample container without a change in sediment concentration or PSD.
- Cross contamination of a sample caused by residual sediments in the system between sample-collection periods should be minimized.

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- The sampler should be capable of sample collection over the full range of sediment concentrations and particle sizes up to about 4 mm (very fine gravel). For example, about 90 percent of the sediment retained in a highway catch basin was less than 4 mm and about 80 percent was less than 2 mm in a highway-runoff study in eastern Massachusetts (Smith, 2000).
- Sample-container volumes should meet minimum sample analysis volume requirements.
- The intake's inside diameter should be maximized to facilitate representative concentrations and PSD of samples [typically 9.5- or 19.0-millimeter- (3/8- or 3/4-inch-) diameter intakes depending on the minimum pumping rate of the sampler used].
- The sampler should be capable of vertical lifts large enough to maintain sample PSD integrity.
- The sampler should be capable of collecting a reasonable number of samples, depending on the purpose of sample collection and the flow conditions.
- Some provision should be made to protect against freezing, evaporation, and dust contamination.
- The sample-container unit should be constructed to facilitate removal and transport as a unit.
- The sampling cycle should be initiated in response to a timing device, flow change, or external signal.
- The capability of recording the sample-collection date and time should exist.
- The provision for operation using alternating current power or direct current (battery) power should exist.

Recent field tests conducted by the USGS in cooperation with the Federal Highway Administration (FHWA) indicate that newer types of automatic pumping samplers meet these criteria, but older vintage (before about 1993) samplers typically do not meet all these criteria (David Owens, USGS, written commun., 2000). For example, several vacuum and peristaltic samplers of post-1993 design used for the field test collected samples with representative PSDs from 20 to 128 µm, but samplers operating on older technologies and construction were not able to collect representative samples when the sampler elevation exceeded the sampler intake elevation by 12 ft or more. In highway systems, it is important to be able to sample larger grain sizes that may be in transport. For example, sediment particle sizes as large as fine gravel (table 4) were collected by an automatic sampler in a highway-runoff study in eastern Massachusetts (Smith, 2000).

Automatic samplers also have technical limitations that must be identified and addressed for representative data-collection and interpretation. Tai and others (1991) and Horowitz and others (1992) provide information and evaluations of automatic-pumping samplers to collect dissolved and solid-phase water-quality constituents, including sediment. Technical limitations may be substantial depending on site and runoff-quality characteristics. Proper site selection and sampling design may compensate for limitations if they are recognized. These limitations include the following:

- Automatic samplers generally are not capable of collecting an isokinetic sample (which is defined as the velocity in the sampler's nozzle being about equal to that of the stream velocity incident on the nozzle because intake velocity is fixed).
- Sample line velocity is reduced with increased elevation between the automatic pumping sampler and the water surface (head), which can compromise measured suspended-sediment concentration (SSC) and PSD values. This effect is caused by the reduced ability of the sampler to lift larger particles (assuming similar particle densities and shapes) over greater heads.
- No currently available samplers are capable of collecting samples at sites where the elevation of the sampler is more than about 28 ft above the sampler intake while maintaining a line speed greater than the minimum of 2.0 ft/s specified by the U.S. Environmental Protection Agency (1982) without the addition of an auxiliary pump (David Owens, written communication, 2000).
- Line lengths greater than about 100 ft may impair the sampler's ability to collect water samples due to line friction.
- Increased intake diameters may be necessary to capture larger grain sizes, but an increase in the inside diameter leads to reduced intake velocities at the same pumping rate.
- Cross contamination of the sample line is a concern and is a function of the line-length (for example, a 1 ft section of a 3/8-inch-diameter tube has an inside surface area of about 0.1 ft^2), and the

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quality of runoff (for example, runoff water that contains other viscous liquids, such as oil and grease) may increase sediment retention.

- Composite samples may be affected by sample volume repeatability, which should be tested for each sampler at each site (David Owens, written commun., 2000).

Samples collected non-isokinetically by automatic samplers may not provide data representative of the mean cross-sectional concentrations and PSDs, particularly when sand-size material is in transport (Edwards and Glysson, 1999). In one study, however, the constituent concentrations of samples collected with an automatic pumping sampler were shown to be similar to those of manually collected, cross-sectionally integrated water-quality samples (Krug and Goddard, 1986). Research is needed to develop relations between data collected by automatic and isokinetic sampling methods in highway and urban drainage systems. In fluvial systems, a depth-integrated sample is required because of potential variations in the cross-sectional distribution of sediment (Guy, 1970). Use of depth-integrating samplers typically requires depths exceeding a foot, and minimum mean flow velocities of about 2 ft/s. In highway and urban drainage structures, however, depth- and width-integrated sampling techniques may not be possible for a number of reasons, such as brief duration of runoff, limited access to the drainage structure, size of the conduit, depths and velocities of water in the conduit, and rapidly varying flows. Also, because of the turbulent flows and rapid mixing characteristic of highway and urban drainage systems, these methods may not be necessary depending on site-specific conditions. If use of the EDI or EWI method to collect samples is possible, samples can be collected over a range of flows on the rising and falling limbs of the hydrograph during different runoff periods to document the difference between the collection of a representative sample and the collection of a sample at a single point by an automatic pump sampler. EDI and EWI samples can be used to develop a cross-section coefficient with concentration values from samples collected with an automatic sampler (Porterfield, 1972) as part of the quality-assurance and quality-control (QA/QC) activities.

To obtain the most reliable and representative data, the automatic sampler intake should be placed at the point at which the concentration approximates the mean sediment concentration for the cross section over a full range of flows. This idealistic concept has great merit, but the mean cross-section concentration almost never exists at the same point or vertical under varying flow conditions. It is even less likely that specific guidelines for locating an intake under given flow conditions at one stage would produce the same intake location relative to the flow conditions at a different stage. For example, there are five possible intake orientations (fig. 4), including (A) horizontal and against flow, (B) horizontal and perpendicular to flow, (C) upward and perpendicular to flow, (D) downward and perpendicular to flow, and (E) horizontal and with flow (Edwards and Glysson, 1999). In laboratory tests of several nozzle orientations, including orientations (C), (D), and (E), Winterstein and Stefan (1986) found orientation (E) to provide the most representative sample in spite of the fact that this is counter-intuitive when considering isokinetic manual sampling techniques (Edwards and Glysson, 1999, p. 14). Winterstein and Stefan (1986) hypothesized that this downstream (with flow) intake minimizes debris accumulation and a small eddy is formed at the intake, which envelops the sand particles and thus allows the sampler to collect a more representative sample of the coarse load than intakes located in other directions with respect to the flow. There are, however, many site-specific issues that must be considered. Therefore, objectives for placing a sampler intake in the flow at any given cross section are as follows:

- Select the intake location so that, if possible, it is submerged for the complete range of flows.
- Identify or install a means to fix the intake at the desired location in flow. The attachment feature and intake should have a high probability of remaining in place at high flows, and should not be prone to collecting debris.
- Make sure the sampler intake is not located where bed material can be drawn into or can bury the intake.
- Locate and configure the sample intake to reduce any potential for debris collection, such as in the downstream direction.
- Sample intake location should be in areas of high velocities and turbulence that offer the greatest potential for mixing, that provide for rapid removal of any particles disturbed during a purge cycle of sample line, (such as downstream of

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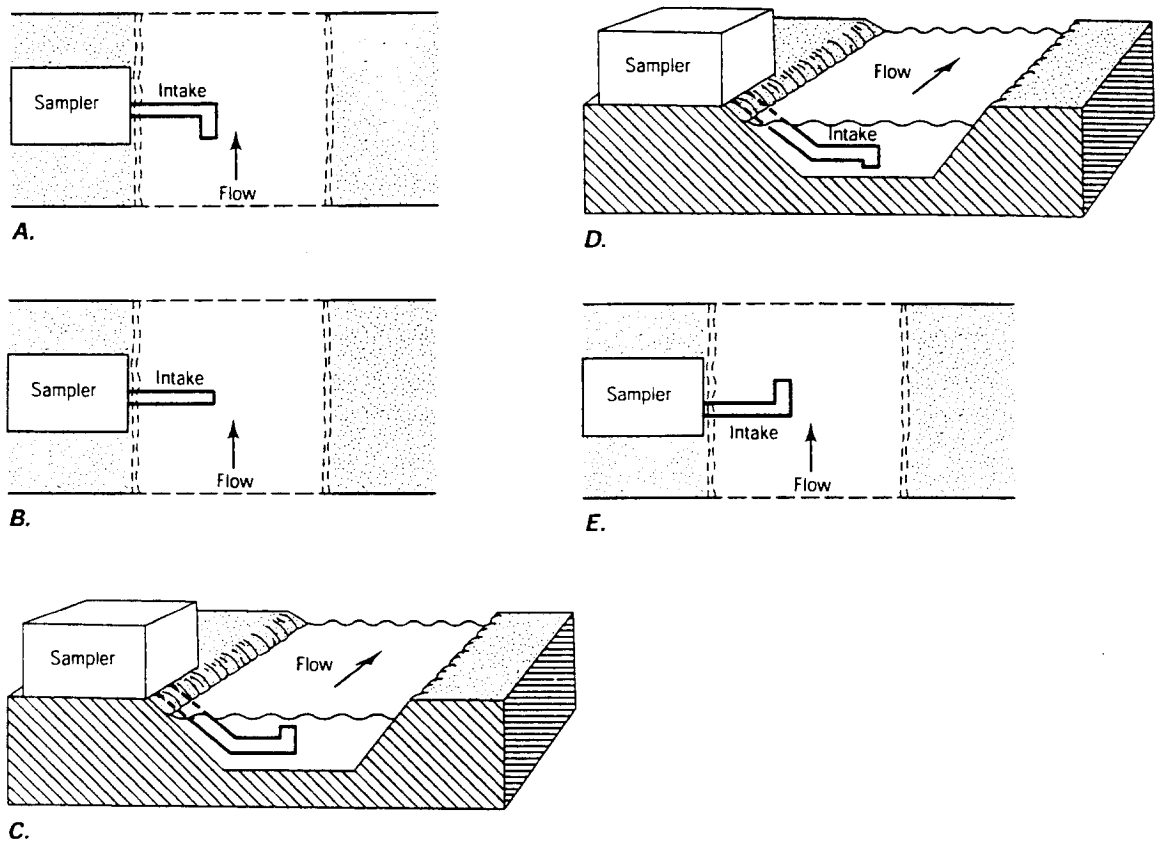

Figure 4. Pumping sampler intake orientations: (*A*) horizontal and against flow; (*B*) horizontal and perpendicular to flow; (*C*) upward and perpendicular to flow; (*D*) downward and perpendicular to flow; and (*E*) horizontal and with flow (modified from Edwards and Glysson, 1999).

storm drainage distribution boxes), or that incorporate static mixing devices just upstream of the sampler intake.

- Mount the tubing with a slope from the intake to the sampler intake to minimize low points in tubing that may retain water and sediment after pumping has ceased. This will reduce the potential for cross contamination between subsequent samples.
- Position one or more intakes as a manifold to collect the most representative (mean) SSC and PSD samples.

Site conditions commonly preclude sampling arrays that meet these guidelines. The investigator should endeavor to install a sampling system that minimizes deviations from these guidelines. It is therefore incumbent upon the investigator to clearly document site-specific conditions and to implement QA/QC measures to quantify the performance of sampling efforts.

Automated pumping samplers can be controlled by a data logger with sampling criteria based on time, stage, rate-of-stage change, or water-quality measurements. An operator can optimize sampling rates in response to changes in expected precipitation volumes during a storm from a remote location, using a communication device such as a cell phone. Gray and Fisk (1992) describe a method for controlling an automatic water sampler based on time, stage, and rate-of-stage-

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change criteria. Their technique is designed to provide an adequate definition of the flood hydrograph to enable reliable computations of daily sediment and associated chemical constituent discharges. Gray and deVries (1984) describe a system for measuring surface runoff and collecting sediment samples from small areas (on the order of hundreds of square feet). Their automatic pumping mechanism splits the sample into 10 equal parts and retains one or more parts as a representative composite sample for the entire runoff period. A technique for controlling an automatic water sampler based on a time-stratified sampling technique is described by Thomas (1985, 1991), and Thomas and Lewis (1993). This capability increases the amount and quality of data derived at a sampling site, and provides a resource to enable the project manager to make informed decisions on allocations of human resources during runoff at one or more sampling sites.

Automatic pumping samplers, however, are not well suited for all sampling sites. The cost, complexity, and logistics (power, communication requirements, and installation) associated with automatic samplers can discourage their use. Also, the sampler intakes need to be positioned in a location with a sufficient cross-sectional area and flow rate to be submerged enough to obtain representative samples. When automatic pumping samplers are impractical, passive automatic samplers may be a viable alternative.

Commercially available automatic samplers are not designed for collection of bedload. Bedload may represent a part of the sediment carried in highway and urban runoff (Bertrand-Krajewski and others, 1993; Waschbusch, 1999). Although newer samplers have collected sediment with particle sizes as large as fine gravel (Smith, 2000), these samplers do not meet specifications for bedload samplers. For example, manual bedload samplers developed by the Federal Interagency Sedimentation Project (2000a; 2000b), such as the BL-84 or the BLH-84 samplers, are designed to collect particles from about 0.25 to 35 mm in diameter using a pressure-difference principle and a nylon mesh screen to retain the sample (Helley and Smith, 1971; Hubbell and others, 1981; Edwards and Glysson, 1999). The necessary tube diameters, pumping rates, and sample volumes required to collect representative samples by automatic samplers may be prohibitive. Research may be needed to develop and test adaptations of automatic pumping samplers to collect representative samples of bedload material for highway runoff studies.

Passive Samplers

Passive automatic samplers are designed to collect a proportion of flow during the time when runoff submerges the sampler intake port(s). Passive samplers generally include a sample intake, an inflow control assembly, a sampling container, and a housing designed to emplace the sampling container and facilitate sample retrieval. A number of automatic types of passive samplers are available. The following are described in greater detail in the noted references.

- A flow splitter described by Clark and others (1981) uses baffles on a steep inclined plane—to cause supercritical flows—(fig. 5) to obtain a representative and flow-proportional sample of an entire storm. Clark and others (1981) indicate that the composite sample reflects the event mean concentrations calculated from a series of discrete samples taken during a monitored storm. To use this sampler, however, the site must be on a fill section of highway so that this sampler may be employed on the highway shoulder at a steep enough slope to function hydraulically. Racin (1995) also describes use of a similar sampler for NPDES monitoring of highway runoff quality in California.
- A catch-basin sampler described by Pratt and Adams (1981) utilizes a series of five conical mesh screens with decreasing slot sizes to capture sediment greater than 1.25, 0.60, 0.40, 0.15, and 0.09 mm, respectively, while allowing runoff water to pass through (fig. 6). Sediments are retained on the screen and concentrations may be estimated by calculating the total flow passing through the screens during the monitoring period.
- A catch-basin sampler described by Ellis and Harrop (1984) uses a number of sieve trays with decreasing mesh sizes from 2.00 to 0.63 mm to capture sediment while allowing runoff water to pass through (fig. 7). This device is similar to the device described by Pratt and Adams (1981) in that sediment loads are retained on the screen and

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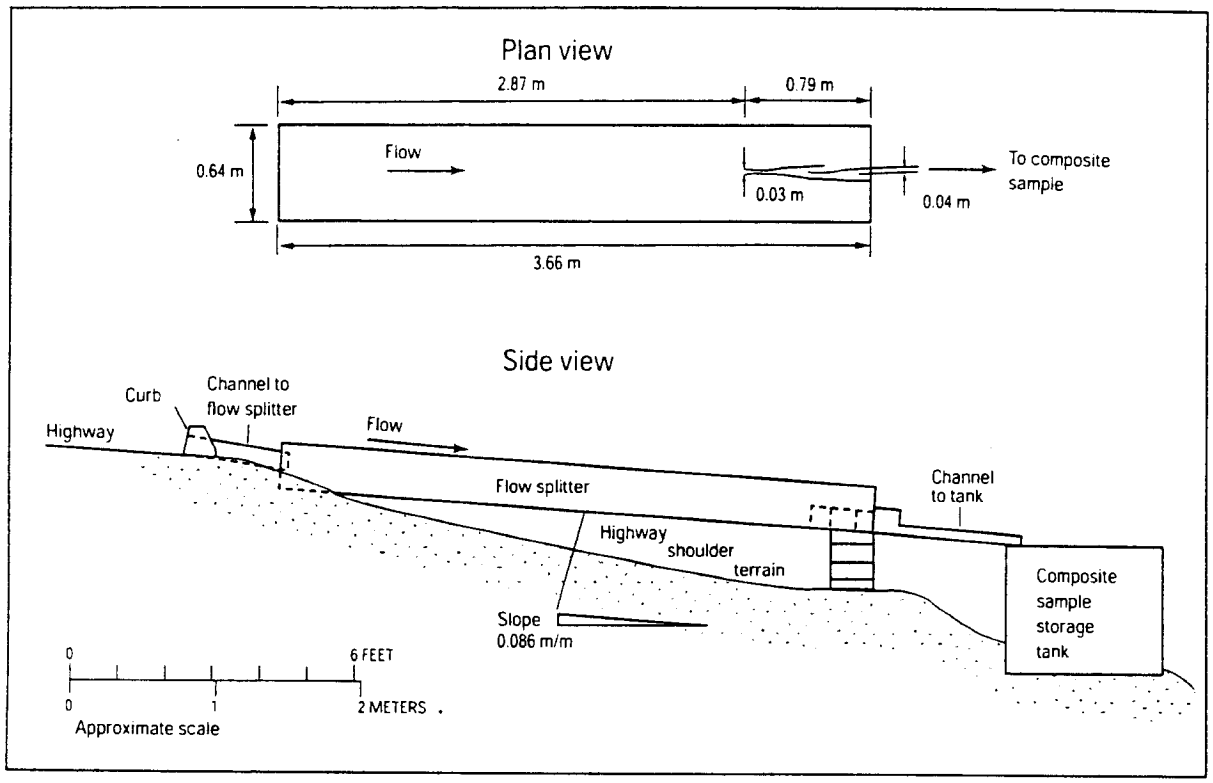

Figure 5. Typical flow splitter (modified from Clark and others, 1981).

concentrations may be estimated by calculating the total flow passing through the screens during the monitoring period.

- A modified single-stage sampler described by Gray and Fisk (1992) is used for passively collecting water samples when the water surface reaches each inlet port in a vertical array of sampling ports (fig. 8).
- A "gully pot" (catch basin) insert described by Spangberg and Niemczynowicz (1992) includes a funnel inlet, water-quality measurement chamber, and a v-notch weir for flow measurement (fig. 9). This device uses a turbidity meter and sampling port to measure turbidity and sediment, respectively.
- A flush mounted sampler (fig. 10*A*) and an "in the pipe" sampler (fig. 10*B*) are described by Dudley (1995). These samplers are designed to collect a water sample by employing a double ball valve so that the sampler is only open during periods of flow immersion and so that the sampler will close once the sample bottle is full.
- A sheet-flow sampler described by Stein and others (1998) is designed to be mounted flush with the pavement (fig. 11). This sampler is normally open to the atmosphere and has a buoyant flap designed to close each inlet port once the receptacle is full.
- A sheet-flow collection system described by Sansalone and others (1998) utilizes a gutter at the pavement edge to concentrates flow through a Parshall flume (to measure flow volume) into a 2,000-liter runoff collection tank (fig. 12). This

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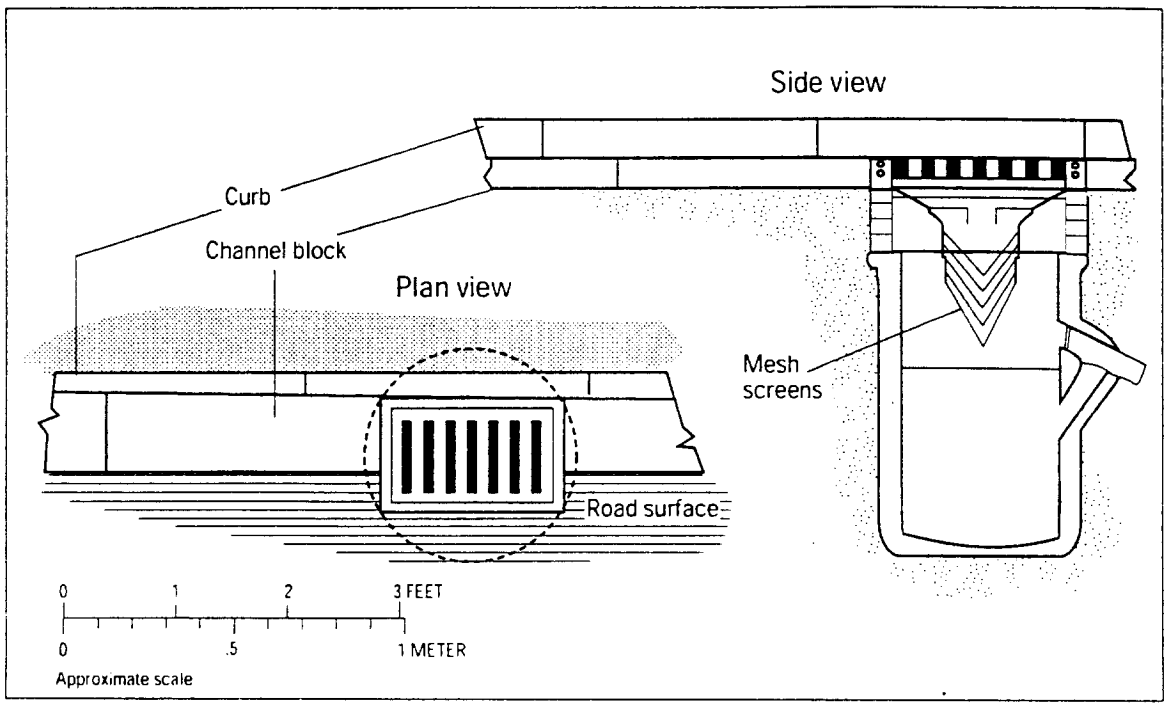

Figure 6. Road washoff material collector for a catch basin (modified from Pratt and Adams, 1981).

passive sampling design also enables use of an automatic pumping sampler by collecting runoff in the flume where flows are concentrated.

- A street-runoff sampler described by Waschbusch and others (1999) is designed to be mounted flush with the pavement for collection of a water sample (fig. 13). This device is normally open and has a setscrew designed to control the inflow volume. Waschbusch and others (1999) also described driveway, lawn, roof, parking lot, and storm-sewer outfall samplers of similar design.

Passive samplers also have technical limitations that must be understood and addressed for representative data collection and interpretation. The quality-control data needed to establish that these samplers perform as expected under the normally harsh highway- and urban-monitoring conditions is not extensive enough to establish comparability and repeatability with other methods. Technical limitations may or may not be substantial depending on site characteristics. Proper site selection and sampling design can compensate for limitations if they are recognized. These limitations include the following:

- Most passive samplers do not have provisions for recording the period of flow sampled.
- Debris buildup on sampler intake(s) can alter the flow of water and sediments completely, precluding sample collection, or partially, thereby affecting the representativeness of samples collected. This may not be apparent, as debris could accumulate and wash off during a single runoff period. However, pumping samplers of relatively recent vintage usually have a purge cycle that may minimize debris buildup.

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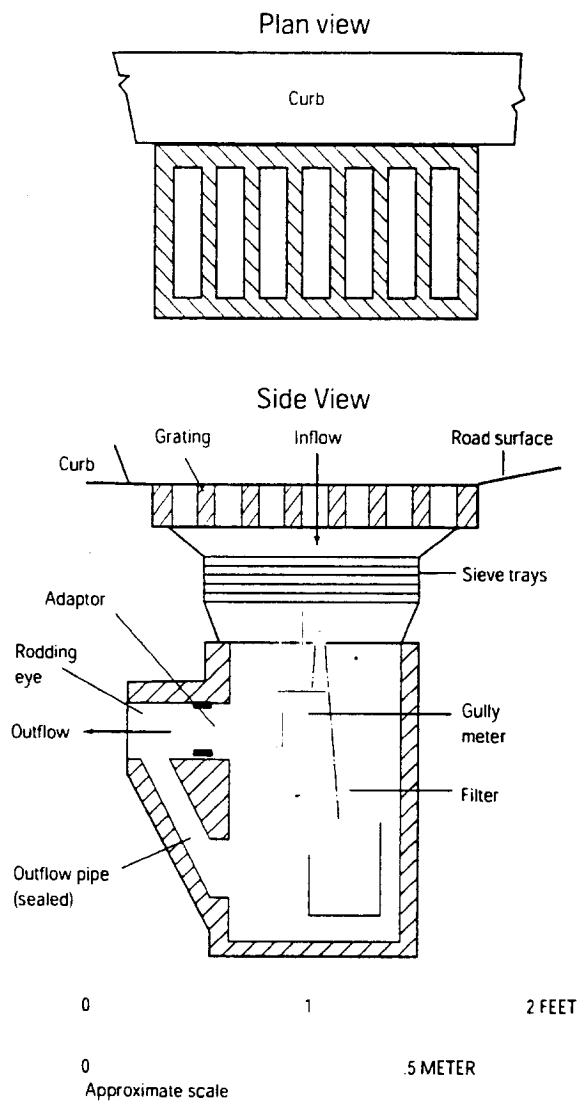

Figure 7. Road washoff material collector for a catch basin (modified from Ellis and Harrop, 1984).

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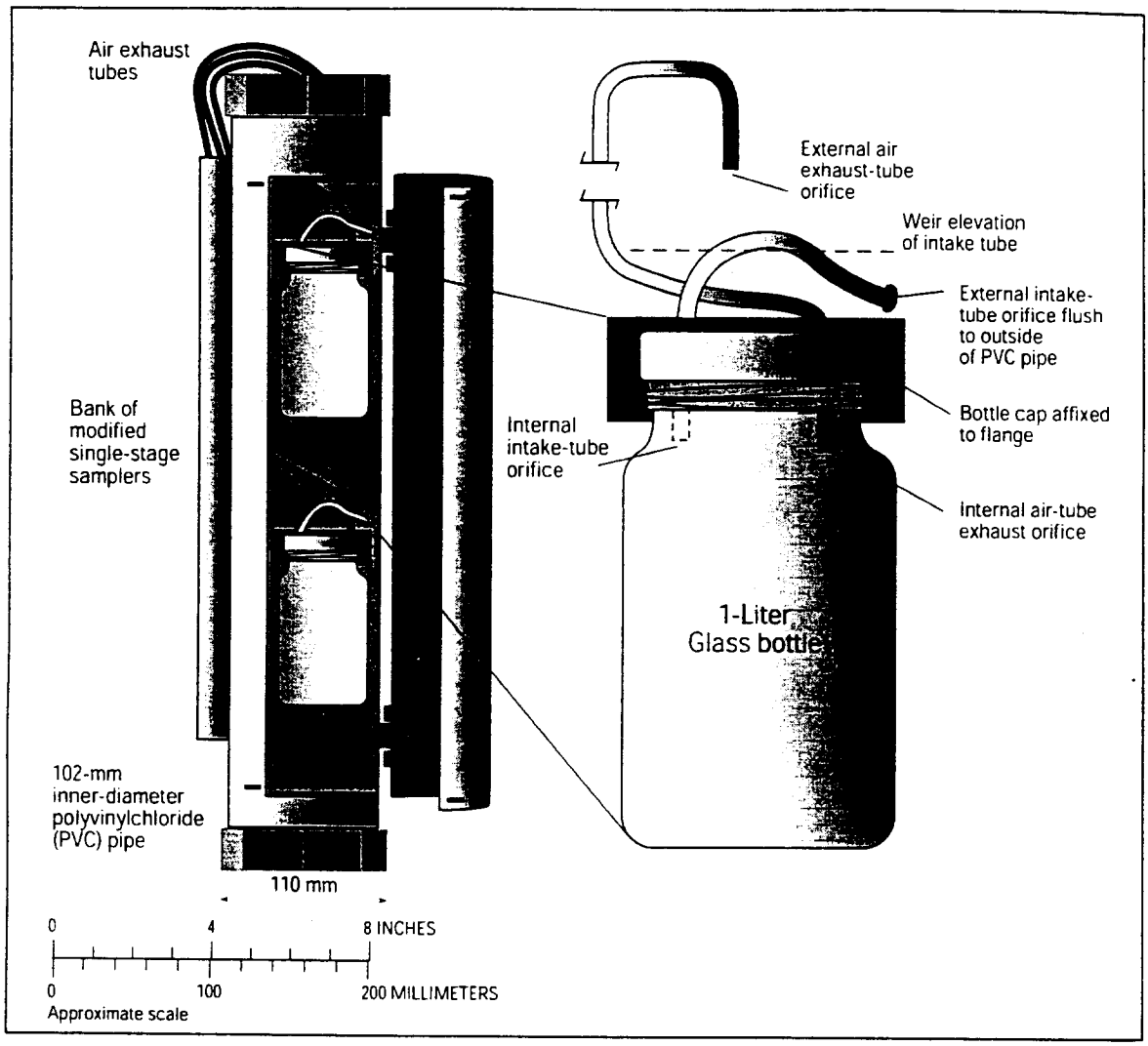

Figure 8. Single-stage sampler (modified from Gray and Fisk, 1992).

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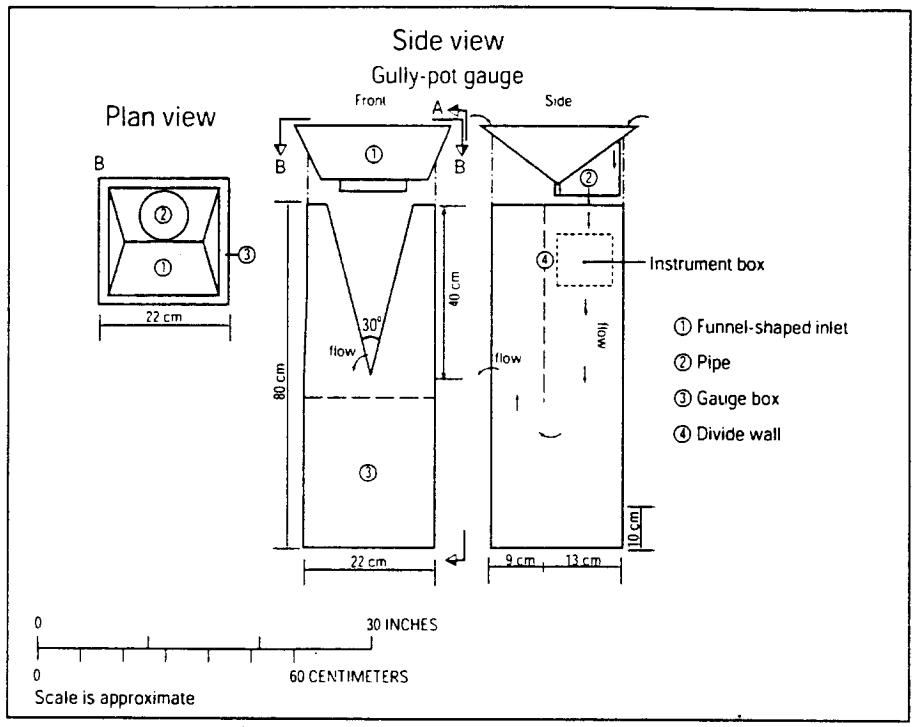

Figure 9. Road washoff material collector for a catch basin (modified from Spangberg and Niemczynowicz, 1992).

- Relatively small contributing areas magnify problems of determining the effective drainage area, and the effects of traffic, bypass flow, and surcharging.
- Relatively small flow-contributing areas also may affect the representativeness of the area sampled and variability in measured concentrations. For example, one piece of rust or tire from a vehicle in a small sampling area could substantially affect a storm load calculated for that area of the highway.
- Samples collected by passive samplers installed on the pavement may be dangerous to retrieve under heavy traffic conditions.
- Passive samplers that are open to the atmosphere may collect debris, sediments, and atmospheric dust blown toward the pavement edge by vehicle action between storms.

To obtain the most reliable and representative data, the passive sampler intake should be placed carefully at a point where sediment concentrations are characteristic of the larger system under study. Therefore, objectives for placing a sampler intake in the flow at any given cross section are as follows:

- Select locations representative of larger study areas.
- Select the intake location so that if possible it is submerged over the complete range of flows or, for single stage sampler(s), the intake is submerged during the intended sampling stage(s).
- Identify or install the device in a position that would not be prone to collecting debris.
- Position one or more intakes to collect the most representative (mean) SSC and PSD samples.

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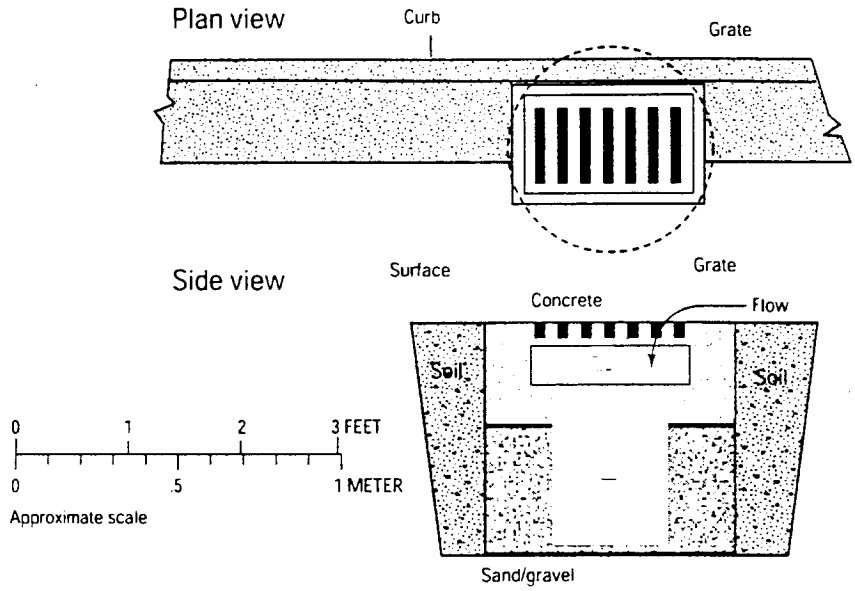
A. Flush mounted sampler, sump and grate set in ground



B. Pipe sampler

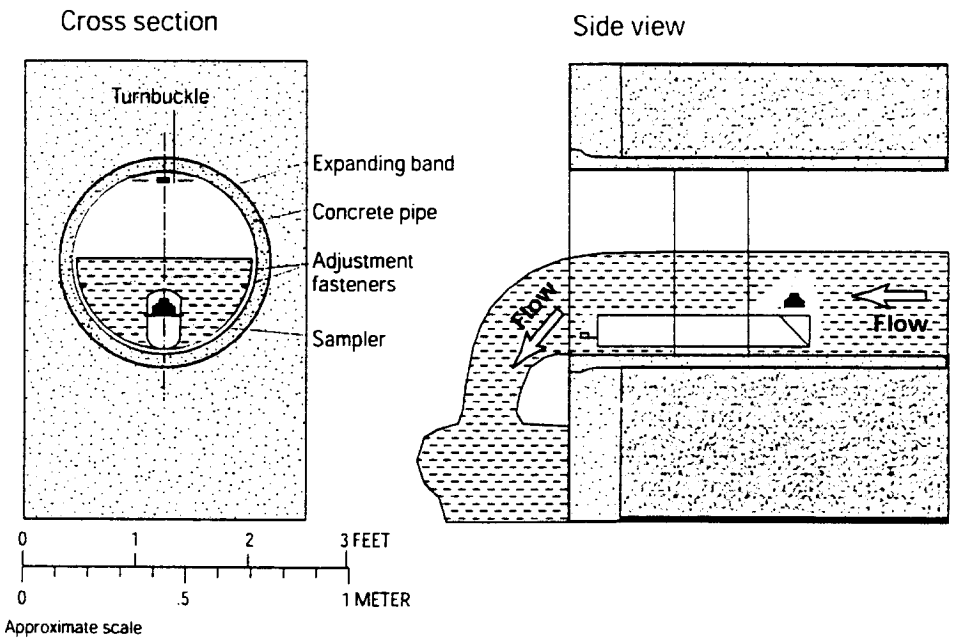


Figure 10. Road washoff material collector for a catch basin and an "in the pipe sampler" (modified from Dudley, 1995).

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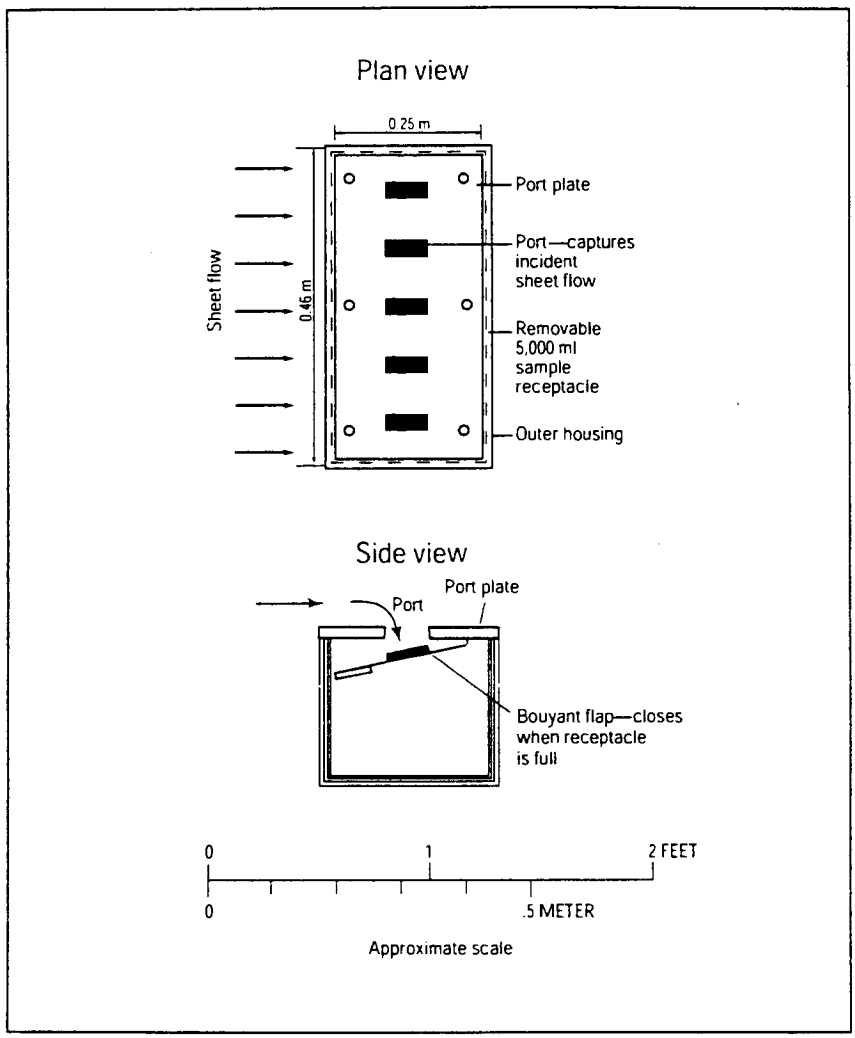

Figure 11. Sheet-flow sampler for a road surface (modified from Stein and others, 1998).

- Emplace multiple samplers to address the problems of small-scale spatial variability and the potential for problems with individual passive samplers.

Site conditions often preclude sampling arrays that meet these guidelines. It is therefore incumbent upon the investigator to clearly document site-specific conditions and to implement QA/QC measures to quantify the performance of sampling efforts.

Some passive samplers are designed for collection of bedload materials. The samplers described by Pratt and Adams (1981); Clark and others (1981); Ellis and Harrop (1984); and Sansalone and others (1998) would be suitable for collection of bedload and suspended sediments. However, research is needed to determine the capture efficiency and other measures of performance for these devices so that the comparability and representativeness of data for highway and urban runoff studies could be assessed. For example, Graczyk and others (2000) compared SSCs of 41 paired samples collected by a single-stage sampler and an automatic pumping sampler and found that mean and median differences (single-stage sampler

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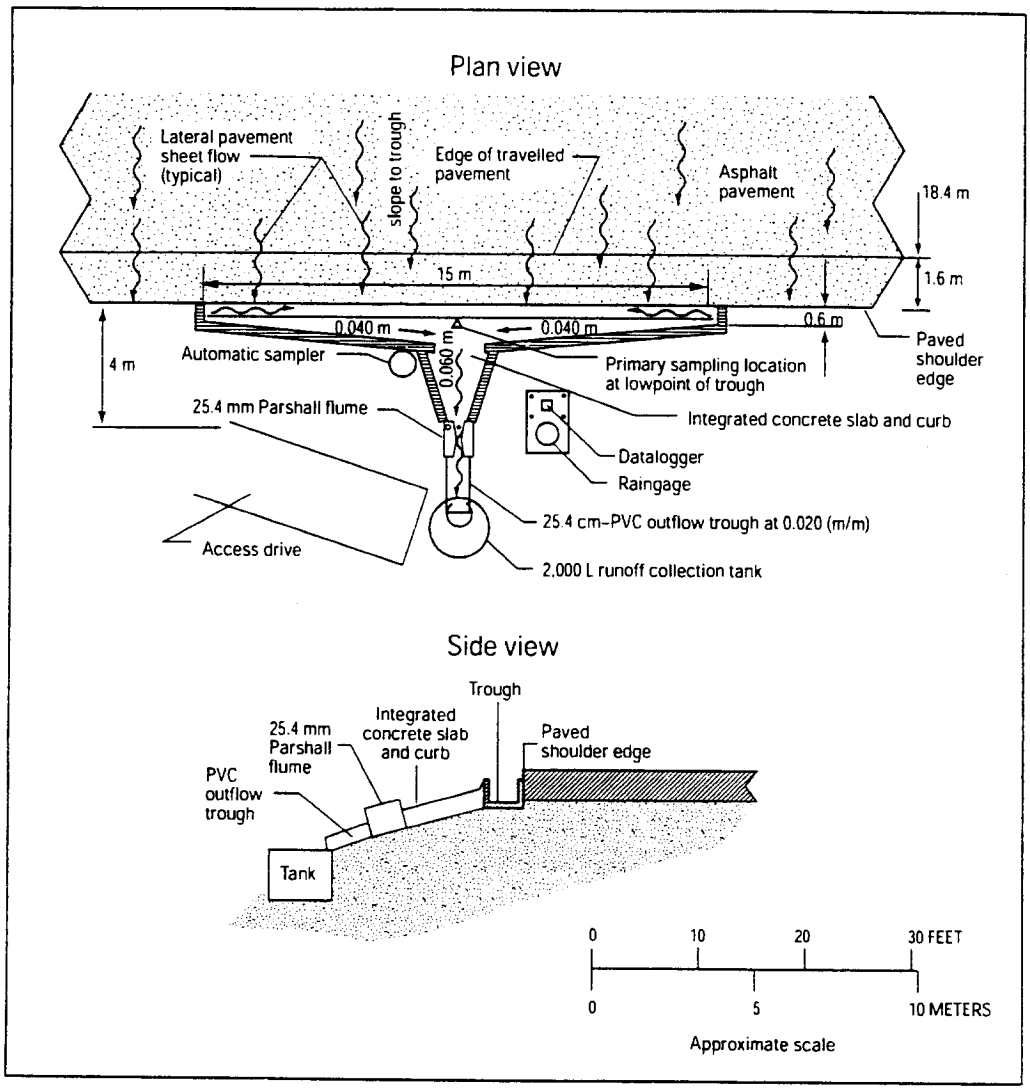

Figure 12. Sheet-flow collection trough for a road (modified from Sansalone and others, 1998).

concentration—automatic pumping sampler concentration) were reasonably similar (14 and 5 mg/L, respectively), but the individual differences had a standard deviation of 133 mg/L and ranged from about -300 mg/L to about 600 mg/L. Therefore, on average, the single-stage samplers may provide representative data but individual sample concentrations collected may vary substantially from samples collected with an automatic pumping sampler (Graczyk and others, 2000).

Indirect Methods for Measuring Sediment

Indirect methods for measuring sediment may be useful as a supplementary and (or) surrogate means to monitoring sediment in runoff. These methods include analysis of available bottom material, measurement of turbidity, and other indirect methods. Each method has benefits and design limitations, which must be recognized and quantified if representative data are to result.

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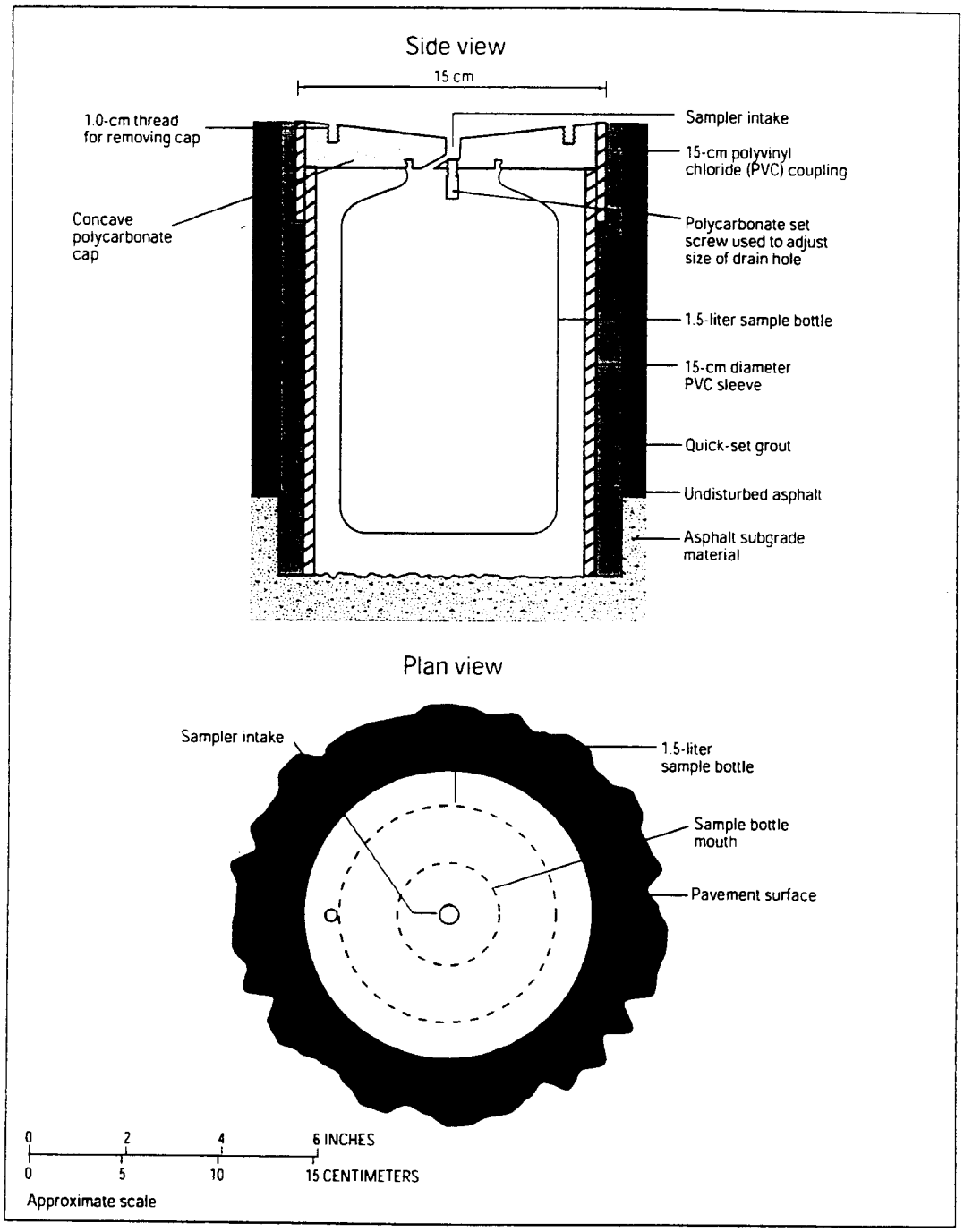

Figure 13. Street-runoff sampler (modified from Waschbusch and others, 1999).

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Bottom Material

Bottom material is the sediment mixture remaining on the bottom of the channel (Edwards and Glysson, 1999). More specifically for highway- and urban-runoff studies, it is the sediment retained on the road, in drainage structures, in structural BMPs, or near drainage outfalls in receiving waters between storms. Repeated analysis of the PSD of bottom material provides information about the sediments transported during runoff events at various flow rates. However, bottom-material samples may not include fine material that moved through the system as washload.

Many of the bed-material samplers designed for fluvial systems may also be suitable for bottom material sampling in runoff conveyances. Edwards and Glysson (1999) describe the samplers developed by the Federal Interagency Sedimentation Project for collecting bed sediments in natural waters. Radtke (1997) lists these and several other bed-material samplers. Yuzyk (1986), Ashmore and others, (1988), Diplas and Fripp (1991), Yuzyk and Winkler (1991), American Society for Testing and Materials (1994), Environment Canada (1994), Shelton and Capel (1994), International Standards Organization (1997b), and Edwards and Glysson (1999) provide bed-material sampling guidelines for subsequent physical and (or) chemical analyses. Kobriger and Geinopolos, (1984) discuss bottom-material sampling in a study of sources and migration of highway runoff pollutants. Materials and methods for collection of bottom-material samples need to be evaluated carefully, especially if these materials will also be used for chemical analysis. As with other methods designed for use in natural (fluvial) systems, the design and implementation of these methods need to be evaluated and QA/QC data need be documented so that the comparability and representativeness of data obtained for highway- and urban-runoff studies may be assessed.

Turbidity

Turbidity is a measure of the light attenuation caused by interference from suspended materials and dissolved materials that produce color. Suspended materials that affect turbidity include organic matter (anthropogenic materials, leaves, and aquatic biota), and natural and anthropogenically derived particulates. Color-producing dissolved materials include iron (as ferric humate) and colloids from the decomposition of organic debris. Turbidity is measured either by a turbidimeter or by an optical backscatterance sensor (OBS) (Downing and others, 1981). Turbidity has been a common surrogate used to estimate SSCs in fluvial systems (Brown and Ritter, 1971; Brown, 1973; Reed, 1978; Beschta, 1980; Smith, 1986; Gippel, 1995; Lewis, 1996 and Schoellhammer and Buchanan, 1998). Turbidity also has been measured in many highway- and urban-runoff studies, including those by Irwin and Losey, (1978), Cramer and Hopkins (1981), McKenzie and Irwin (1983); Dupuis and others (1985), Schiffer (1989); Spangberg and Niemczynowicz (1992); and Barrett and others (1996).

Turbidity can be measured to provide real-time estimates of SSCs in flowing waters. To date, several researchers (Reed, 1978; Lewis, 1996; and Eychaner, 1997; Buchanan and Schoellhamer, 1998; Schoellhamer and Buchanan, 1998) have used continuous turbidity data from turbiditimeters or OBS as a surrogate for SSCs. Spangberg and Niemczynowicz (1992) used turbidity measurements to estimate sediment runoff from a parking lot on a 10-second interval. They found substantial variations—related to variations in flow—at time scales on the order of about one minute. In many highway- and urban-runoff studies, however, turbidity is at best a qualitative indication of sediment concentration because measured turbidity depends on many factors, including the PSD of sediments, the quality and maintenance of the probe, the effects of degree of fouling (trash, sediment, and biota) and temperature on the probe. Laboratory analysis of turbidity and SSC data for 1,135 runoff samples collected in a highway drainage pipe in eastern Massachusetts (Smith, 2000) indicates that the relation between measured values is qualitative over the full range of measured sediment concentrations. For example, at a measured turbidity of 100 nephelometric turbidity units (NTU), the SSC ranged from about 70 to 2,000 mg/L, and at a turbidity of 1,000 NTU the SSC ranged from about 700 to 3,000 mg/L (fig. 14). These data do not include the additional variability of measuring turbidity in a harsh field environment that would further reduce the reliability of any quantitative relation developed between turbidity and SSC.

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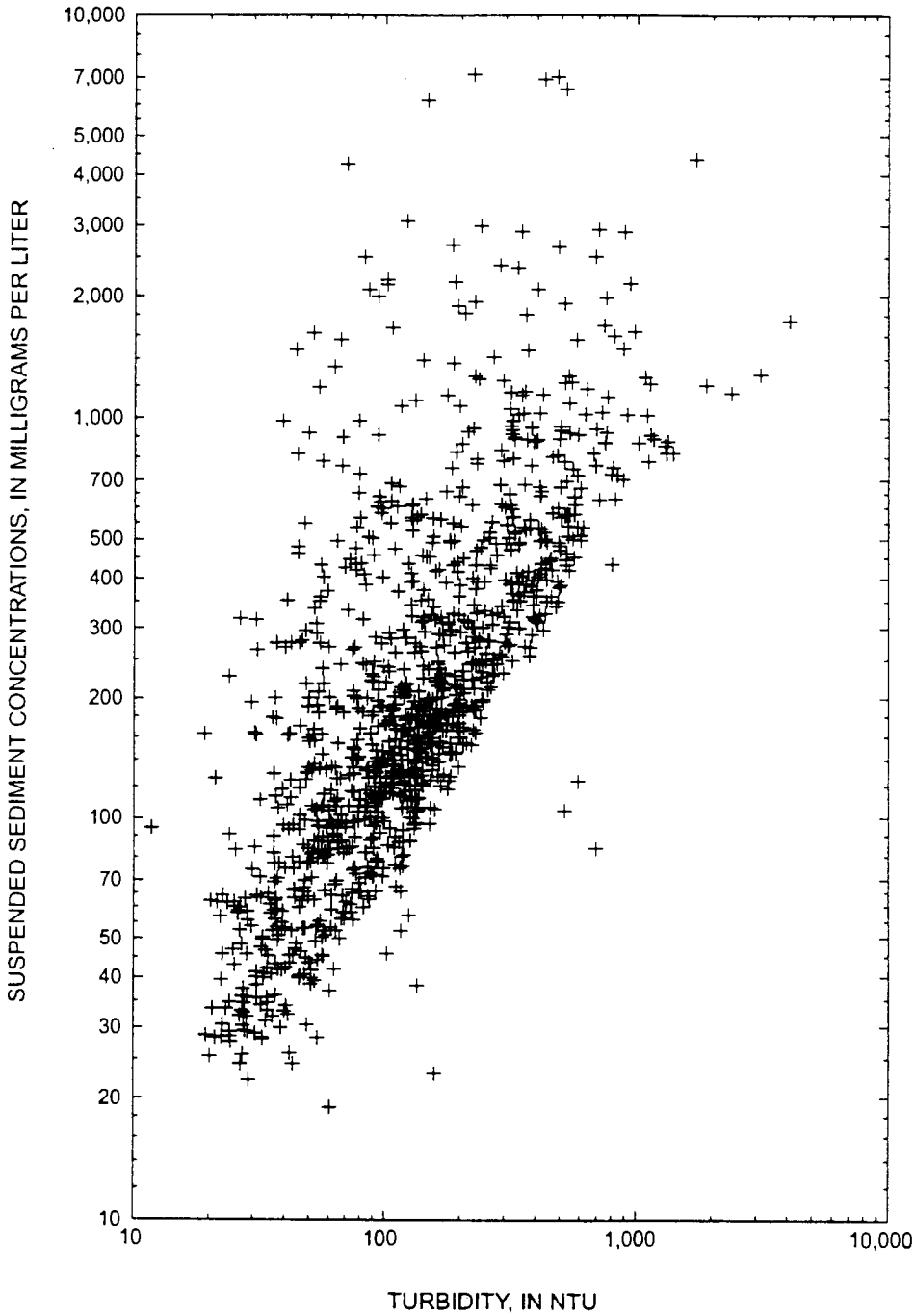

Figure 14. The relation of suspended-sediment concentration to turbidity in highway-runoff samples collected along I-93 in Boston, Massachusetts, 1999–2000 (data from Smith, 2000).

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Turbidity as an optical measurement is easily fouled by oil and grease, biofilm, and other materials found in highway runoff. Also, field measurements are affected by temperature, color, bubbles, and larger particles, which may disproportionally influence turbidity in the small field of view of the instrument.

Calibration and maintenance of in-situ turbidimeters and OBS, can be expensive and time consuming. Each instrument must be calibrated periodically on-site with standards. The accuracy of these instruments often tends to vary in one direction, or "drift," as the sensor becomes fouled with sediment or biota. In spite of these problems, Lewis (1996), and Buchanan and Schoellhamer (1998) recently demonstrated improved accuracy in measuring continuous turbidity data to calculate suspended-sediment discharges in fluvial systems. Highway and urban conveyances, however, can be more challenging because of intermittent flows; large variations in the concentrations and PSDs of sediment, and because of the difficult monitoring environment. As with other sediment monitoring methods, use of turbidity data needs to be evaluated and QA/QC data need be documented so that the comparability and representativeness of data obtained for highway- and urban-runoff studies may be assessed.

Other Indirect Methods

Skinner and others (1986), Ficken (1986), and Skinner and Szalona (1991) describe other surrogate measurement techniques to infer SSCs. These include a transmissometer, x-ray particle size analyzer, ultrasonic suspended-solids meter, radioisotope gage, vibrating U-tube fluid density tube, vibrating straight tube, and the plummet gage. They report limited successes with these technologies in estimating SSCs in fluvial systems, and none is currently being deployed by the USGS in large-scale monitoring programs. Wren and others (2000) describe emerging technologies as surrogates for measuring SSCs. A suite of emerging technologies for the measurement of suspended sediment, bed material, and bedload, is described by Gray and Schmidt (1998). New technologies that measure suspended sediment and (or) bed topography include acoustic (Kuhnle and others, 1998; Mueller, 1998; Derrow and others, 1998; Garcia and Admiraal, 1998), optic (Muste and Kruger, 1998; Schmidt, 1998a, 1998b), fluid density (William Fletcher, Design Analysis Associates, Inc., written commun., 2000; Dirk de Hoop, Hope Hydrology, written commun., 2000), satellite (Chavez, 1998), laser techniques (Yogi Agrawal, Sequoia Scientific, Inc., written commun., 1998), and electro-mechanical techniques (Jobson, 1998). These technologies are being developed for fluvial systems and show some promise to automate and (or) improve the quality of sediment data collection in that environment. These techniques may also be applicable for future highway- and urban-runoff studies including the monitoring of BMP structures and receiving waters. Currently, the difficult highway- and urban-monitoring environment may preclude use of these devices, but future technical developments may improve the potential utility of these methods. Mineral magnetic techniques, however, may be useful for source identification and may be used to follow the transport and sequencing of surface sediments through a stormwater conveyance systems (Beckwith and others, 1990). As with other methods designed for use in natural (fluvial) systems, the design and implementation of these methods need to be evaluated and QA/QC data need to be documented so that the comparability and representativeness of data obtained for highway- and urban-runoff studies may be assessed

SAMPLE-PROCESSING METHODS

Appropriate sample processing methods are determined by the characteristics of the water sampled and by the analytical and interpretive methods used for data reduction. Water-quality data for highway and urban runoff are generally reported as event mean concentrations (EMC) to provide summary values that can be used to compare measurements from individual runoff periods at a site or from populations of storms between sites. Theoretically, the EMC for suspended sediments is the cumulative storm load (mass) of suspended sediment divided by the total runoff volume for the storm period (event) (Driscoll and others, 1991). An EMC may be determined by collecting a bulk sample, by physically compositing a number of discrete samples, or by mathematically calculating a flow-weighted

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composite value from analysis of multiple discrete samples taken during the runoff period. The composite sample can be obtained manually from discrete stormwater samples by methods described by Gray and Fisk (1992), or automatically by methods described by Heaney and Huber (1979).

Each sample type has certain pre-analysis processing requirements that may affect measured sediment concentrations. Large bulk samples require homogenization and subsampling to produce an aliquot suitable for laboratory analysis and (or) for concurrent analysis of other water-quality constituents. When discrete samples are physically composited, the resulting bulk volume must also be homogenized and representatively subsampled to produce an aliquot suitable for laboratory analysis and (or) for concurrent analysis of other water-quality constituents. Discrete samples, however, need not be homogenized and subsampled unless laboratory analysis for concurrent analysis of other water-quality constituents is necessary. This requirement can be avoided by collection of duplicate discrete samples for analysis of sediment and other water-quality constituents.

Homogenization

Homogenization is necessary when subsamples will be extracted from an aliquot for analysis. Also, large bulk-samples collected by passive samplers such as described by Clark and others (1981) need to be homogenized and subsampled to obtain volumes that are feasible for laboratory analysis. The objectives of a homogenization process are to provide a uniform distribution of sediment concentrations and PSDs in each subsample extracted. Homogenization is accomplished by imparting kinetic energy to the solution to uniformly suspend all particles in solution. The presence of particles larger than medium sands (about 0.25 mm, table 4) increases difficulties associated with obtaining representative subsamples. Additionally, it may be impossible to evenly distribute several sediment grains throughout a sample container for representative subsampling.

Before 1976, USGS guidelines on manual sample splitting required compositing the water sample into a large, clean jug or bottle, and shaking it for uniform mixing (U.S. Geological Survey, 1976). In 1976, the 14-liter churn splitter, which utilizes a large plunger to mix a composite water sample was introduced to facilitate the withdrawal of a representative subsample of a water-sediment mixture (Capel and Larson, 1996; Wilde and others, 1998). Demonstrating the comparability of the homogenization process among different samples is important to establish that sediment subsamples are representative and that sediments included in each subsample used for chemical analysis are comparable. It is therefore incumbent upon the investigator to use consistent homogenization protocols, to clearly document site-specific conditions (such as the range of concentrations and PSDs) and to implement QA/QC measures to quantify the performance of this sample processing method.

Subsampling Water-Sediment Mixtures

Samples of water-sediment mixtures are sometimes subsampled, or split into multiple parts to enable different analytical determinations on the subsamples. The validity of data obtained from subsamples depends on the comparability of selected constituent concentrations in the subsample to those in the original sample. Subsamples tend to have larger constituent variances than the original, and may also be biased. Subsampling should be avoided unless it is necessary to achieve the ends of the sampling program. Currently, the 14-liter churn splitter is commonly used to collect subsamples for analysis of sediment concentrations (Capel and Larson, 1996; Wilde and others, 1998). The cone splitter, developed to split water samples for suspended sediment and other water-quality constituents into ten equal and representative aliquots, was introduced for wide-scale use in 1980 (Capel and Nacionales, 1995; Capel and Larsen, 1996). Results of tests on the sediment-splitting efficiency of the churn and cone splitters were published in 1997 (U.S. Geological Survey, 1997). The churn splitter was approved for providing subsamples when the original sample's sediment concentration is less than 1,000 mg/L at mean particle sizes less than 0.25 mm. At SSC concentrations of 10,000 mg/L or more, the bias and precision of sediment concentrations in churn splitter subsamples are considered unacceptable (U.S. Geological Survey, 1997; Wilde and others, 1999b). The cone splitter was approved for providing subsamples at sediment concentrations up to 10,000 mg/L at mean particle sizes less than 0.25 mm. The test data suggest that the

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cone splitter's acceptable concentration range exceeds 10,000 mg/L, and may approach 100,000 mg/L at PSDs less than 0.25 mm.

The usefulness of data obtained from subsamples depends on their comparability of selected constituent concentrations to those in the original sample. Demonstrating the comparability of the subsampling process is important to establish that sediment subsamples are representative and comparable. It is therefore incumbent upon the investigator to select consistent subsampling protocols, to clearly document site-specific conditions (such as the range of concentrations and PSDs) and to implement QA/QC measures to quantify the performance of this sample processing method.

SAMPLE-ANALYSIS METHODS

Representative analysis of concentrations and physical characteristics of sediment in highway and urban runoff is an integral step toward assuring data quality. Methods for determining the physical characteristics of sediment that are pertinent to the study of highway and urban runoff include PSD, specific gravity, settling velocity, and the organic content of sediment.

Measurement of Sediment Concentration

Virtually all solid-phase concentration values determined in the United States are obtained by one of two analytical methods: the suspended-sediment concentration (SSC) method (American Society for Testing and Material, 2000) and the total suspended solids (TSS) method (American Public Health Association and others, 1995). Analytical methods used to produce SSC and TSS data differ; however, the terms are often used interchangeably to describe the concentration of solid-phase material suspended in a water-sediment mixture, usually expressed in milligrams per liter (G.E. Granato, USGS, oral commun., 1999; James, 1999). Extensive review of highway- and urban-runoff water-quality literature indicate that these studies commonly do not define the analysis method, but almost all describe the total concentration of suspended solid-phase material in terms of TSS, regardless of the method used (G.E. Granato, USGS, written commun., 2000). For example, the draft report, "Proposed Sediment Total Maximum Daily Load for Stekoa Creek, Georgia" (U.S. Environmental Protection Agency, Region 4, written commun., 2000) uses "regional TSS data" that are compiled from U.S. Geological Survey records; the TSS data referred to are actually SSC data. Buchanan and Schoellhamer (1998) refer to sediment data collected as "suspended-solids concentration data" for San Francisco Bay monitoring efforts. These data would more appropriately be referred to as SSC, because the total water-sediment mass and all sediment are measured in the analysis (Alan Mlodnosky, USGS, oral commun., 1999).

Use of SSC and TSS in load calculations can produce substantially different results (Glysson and others, 2000, 2001; Gray and others, 2000, 2001a; Gordon and others, 2001). Although these methods are often expected to produce comparable results, recent research indicates that there are systematic differences between methods (Glysson and others, 2000, Gray and others, 2000).

The SSC method (American Society for Testing and Materials, 2000) uses standardized procedures and equipment to measure all of the sediment and the net weight of the water-sediment mixture to calculate concentration. Three analytical methods are used to produce SSC data: Evaporation, filtration, and wet-sieving filtration of the entire sample volume received by the laboratory. The evaporation method is applicable for all concentrations; if the dissolved-solids concentration exceeds about 10 percent of the sediment concentration, an appropriate correction factor must be applied to the suspend-sediment concentration value derived by the evaporation method because these solids are included in the analysis. The filtration method is used only on samples with concentrations of sand-size material (diameters greater than 0.062 mm) less than about 10,000 mg/L and clay-size material concentrations of about 200 mg/L or less. No dissolved solids correction is needed. The wet-sieve-filtration method yields a concentration for the total sample, a concentration of the sand-size particles, and a concentration for the silt- and clay-size particles. A dissolved-solids correction may or may not be needed, depending on the type of analysis done on the fine fraction and the dissolved-solids concentration of the sample. These three methods are virtually the same as those used by USGS sediment laboratories and described by Guy (1969). USGS sediment laboratories, however, use the Whatman grade 934AH, 24-millimeter-diameter filter

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for purposes of standardization. Each method includes retaining, drying at 103° to 105°C, and weighing all of the sediment in a known mass of a water-sediment mixture. The USGS analyzes sediment samples using methods described by Guy (1969), Matthes and others (1991), Knott and others (1992, 1993), and U.S. Geological Survey (1998a; 1998b; 1999a). Most of these methods were developed by the Federal Interagency Sedimentation Project, were approved by the Technical Committee (Glysson and Gray, 1997), and are used by most Federal agencies that analyze fluvial sediment data.

According to Gray and others (2000), all three SSC test methods have analytical uncertainties (precision and bias) on the order of plus or minus:

- 6 to 40 percent at low concentrations (about 10 mg/L),
- 2 to 20 percent in midrange concentrations (from 100 to about 1,000 mg/L), and uncertainties decrease proportionally with increasing concentrations (greater than 1,000 mg/L).

Tests of SSC quality-control samples by sediment laboratories participating in the USGS National Sediment Laboratory Quality Assurance Program (Gray and others, 2000; 2001a; 2001b) provide estimates of bias and variance associated with sediment data. The median concentration bias for all participating laboratories is -1.83 percent, and the 25th and 75th percentile values are -4.39 and 0.00 percent, respectively (Gray and others, 2000). The bias primarily reflects a loss of some sediment, such as through a filter or an inability to weigh accurately very small amounts of sediment. Gordon and others (1999) show that the concentration bias is largest at smaller concentrations, and very small at concentrations exceeding about 2,000 mg/L.

The TSS analytical method (American Public Health Association and others, 1995) originated as an analytical method for wastewater samples. The fundamental difference between SSC and TSS analytical results stems from preparation of the sample for subsequent filtering, drying, and weighing. In contrast with the SSC analytical method, the TSS method requires analysis of a subsample extracted from the original. The standard method requires a subsample, or aliquot volume of 100 mL, unless more than 200 mg of residue is expected to collect on the filter, in which case a proportionally smaller volume is removed. The standard specifies that a subsample be extracted from the original water sample by pipette while the entire sample volume is being mixed with a magnetic stirrer. The subsample is filtered, and the filter and contents are removed and dried at 103° to 105°C, and weighed. No dissolved-solids correction is required and the standard provides no indication of the size of particles used in the testing for the method. The percentage of sand-size and finer material cannot be determined using the TSS method.

In practice, TSS data are produced by a number of variations to the processing methods described in the American Public Health Association and others (1995). For example:

- For the collection of TSS samples from the Chesapeake Bay Program, field staff pump water from a specified depth into a plastic gallon container. The container is vigorously shaken, and 200–1,000 mL of the water-sediment mixture is poured for field filtering and subsequent analysis. (Mary Ley, Interstate Commission on the Potomac River Basin, the State of Maryland, and the Commonwealth of Virginia, written commun., 2000).
- A State government laboratory in Virginia produces TSS data by vigorously shaking the sample and pouring the sample into a crucible for subsequent analysis. All of the sample is poured into the crucible unless "there is a lot of suspended material," in which case only part of the sample is poured (Lori Sprague, U.S. Geological Survey, written commun., 1999).
- One laboratory analyzed TSS quality-control samples using Method 2540D of the American Public Health Association and others (1995), with the following variation: The sample is shaken vigorously and a third of the desired subsample volume is decanted to a secondary vessel. This process is repeated twice to obtain a single subsample for subsequent filtration, drying, and weighing.

The reduction in TSS data comparability by variations in protocols used is not limited to lack of consistency in processing and analytical methods. According to James (1999), there is generally no agreed upon definition of TSS in regard to stormwater runoff, in part because the settleable part of TSS is not reported in most stormwater studies.

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If a sample contains a substantial percentage of sand-size material, stirring, shaking, or otherwise agitating the sample before obtaining a subsample will rarely produce an aliquot representative of the sediment concentration and PSD of the original sample. This is a by-product of the rapid settling properties of sand-size material, compared to those for silt- and clay-size material, given virtually uniform densities as described by Stokes' Law. Aliquots obtained by pipette might be withdrawn from the lower part of the sample where the sand concentration tends to be enriched immediately after agitation, or from a higher part of the sample where the sand concentration is rapidly depleted. Additionally, the physical characteristics of a pipette used to withdraw an aliquot can introduce bias in the subsample (Gray and others, 2000).

The physical characteristics of a pipette used to withdraw an aliquot, or subsample, can introduce additional errors in subsequent analytical results. The tip opening of pipettes recommended for use is about 3 mm in diameter (Kimble-Contes, Inc., 2000). The upper limit of sand-size particles, which is expressed as the median diameter, is 2 mm (Folk, 1980). A natural sediment particle's long axis is almost always larger than its median diameter and can be substantially larger. Hence, a single coarse-grained sand particle or multiple sand-size particles, particularly when present in large concentrations, may clog a 3-millimeter tip pipette under suction.

Subsampling errors are hardly limited to use of a pipette to withdraw a sample. Methods that include pouring of a subsample can introduce additional errors in subsequent analytical results. This is because, based on Stokes' Law, subsamples obtained by pouring a sand-rich water-sediment mixture likely will be deficient in sand-size material due to settling in the original sample. Fine-material concentrations will not normally be altered by the removal of an aliquot.

Bias in results produced by the SSC and TSS analytical methods tends to become apparent when sand-size material exceeds about a quarter of the sediment mass in the sample (Gray and others, 2000). This has important ramifications with respect to transport calculations. Solid-phase concentrations tend to increase with discharge for a stream, as does the percentage of sand-size material in transport. High flows tend to be inordinately influential in mass transport. Bias in TSS data would probably be largest at higher flows, and therefore transport calculations based on TSS data are prone to be biased. Glysson and others (2000; 2001) indicate that transport estimates using TSS data can be orders of magnitude in error. Gray and others (2000) show an example of a stream at low flow where instantaneous-value sediment discharges calculated from SSC data are more than an order of magnitude larger than those calculated from TSS data at similar flow rates. Glysson and others (2000; 2001) conclude that there appears to be no simple, straightforward way to relate TSS and SSC data unless pairs of TSS and SSC results are available for a site.

Part of the problem may be attributable to the origin of the TSS method and subsequent changes in the types of water for which it is recommended for use. The American Public Health Association and others (1971) intended the Total Suspended Matter Method (a precursor to the TSS method) to be suitable for "wastewaters, effluents, and polluted waters." In 1976, the American Public Health Association and others deemed the Total Suspended Matter Method as suitable for "residue in potable, surface, and saline waters, as well as domestic and industrial wastewaters in the range up to 20,000 mg/L." Gray and others (2000) conclude that the TSS analytical method is being misapplied to natural-water samples, and that the TSS method is fundamentally unreliable for that purpose. Additionally, the percentage of sand-size and finer particles can not be determined as a part of the TSS method, whereas it can be determined as a part of the SSC method.

The USGS (2000) policy on the collection and use of TSS data establishes that TSS concentrations and resulting load calculations of suspended material in water samples collected from open channel flow are not appropriate. The TSS analytical method can result in unacceptable large errors and is fundamentally unreliable to determine concentrations of suspended material in open channel flow. Therefore, based on these issues, the USGS standard for determining concentrations of suspended material in surface-water samples continues to be the SSC method of analysis (USGS, 2000).

These findings are directly relevant to sediment-load estimates in runoff from highways and urban areas. Highway and urban runoff tends to be rich in sand-size material (table 3); hence, TSS analytical results from samples collected may be substantially biased. As described previously, load estimates using TSS data can deviate by orders of magnitude from those calculated from SSC data.

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To examine the applicability of TSS data in highway and urban runoff, analyses of paired SSC and TSS samples collected by the USGS from studies of highway sediments along I-93 in Boston, Mass. (Smith, 2000), and along I-894 in Milwaukee, Wisc. (Waschbusch, 2000), were examined (fig. 15). As with the natural water samples examined by Glysson and others (2000) and Gray and others (2000), results from the TSS analytical method have a substantial negative bias when compared to the SSC method. These data sets (Smith, 2000; Waschbusch, 2000) also indicate that the sand-size fractions can be substantial, and that the grain-size distributions vary considerably between samples within a storm and also vary considerably among storm-runoff periods (fig. 16). These TSS data are not representative of sediment loads from highways and cannot be quantitatively adjusted to produce SSC estimates because of the large variability in grain-size distributions from storm to storm. Analyses of these data sets, obtained from a coastal site and an interior sites in the conterminous United States, indicate substantial differences between TSS and SSC in highway-runoff data that are consistent with those reported by Glysson and others (2000) and Gray and others (2000). It is therefore incumbent upon the investigator to select consistent subsampling protocols, to clearly document

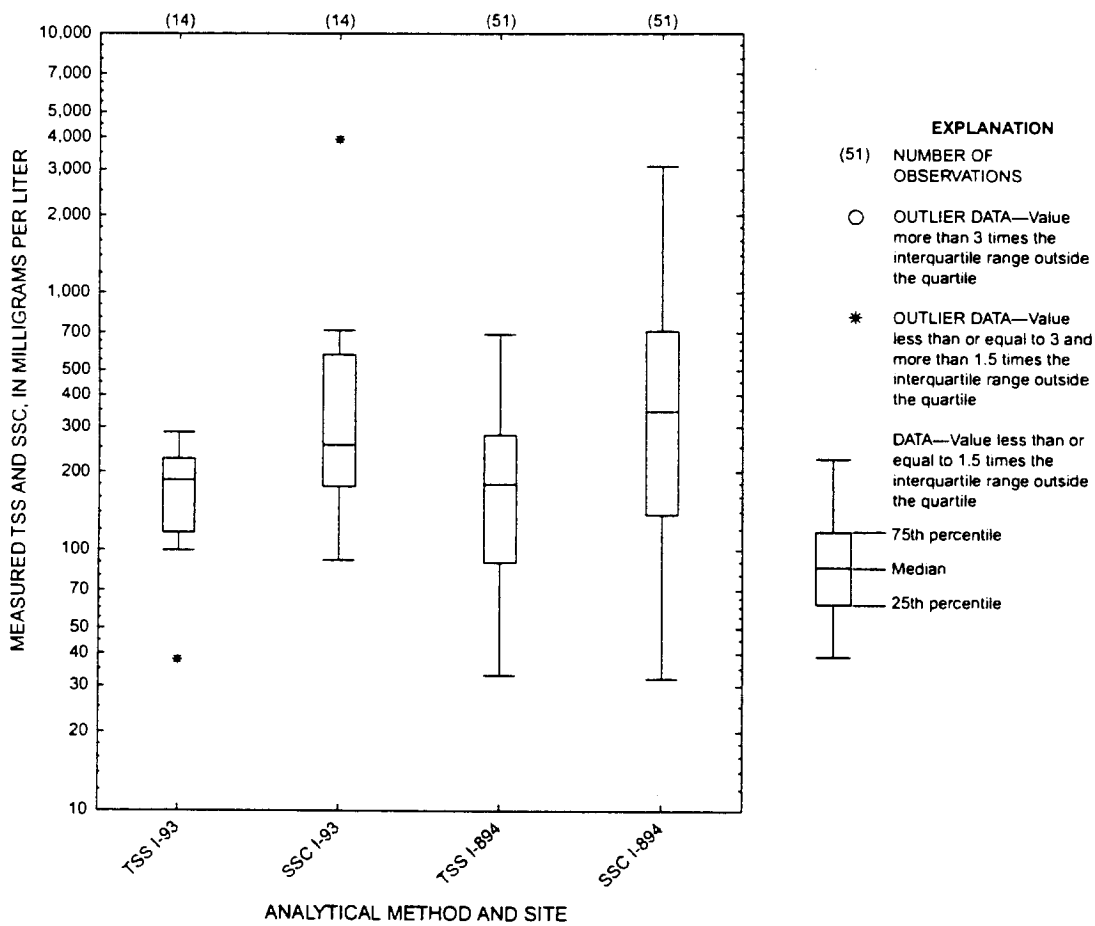

Figure 15. Distribution of measured total suspended solids (TSS) and suspended-sediment concentration (SSC) in highway-runoff samples collected along I-93 in Boston, Massachusetts, and I-894 in Milwaukee, Wisconsin, 1999–2000 (data from Smith, 2000 and R.J. Waschbusch, U.S. Geological Survey, written commun., 2000, respectively).

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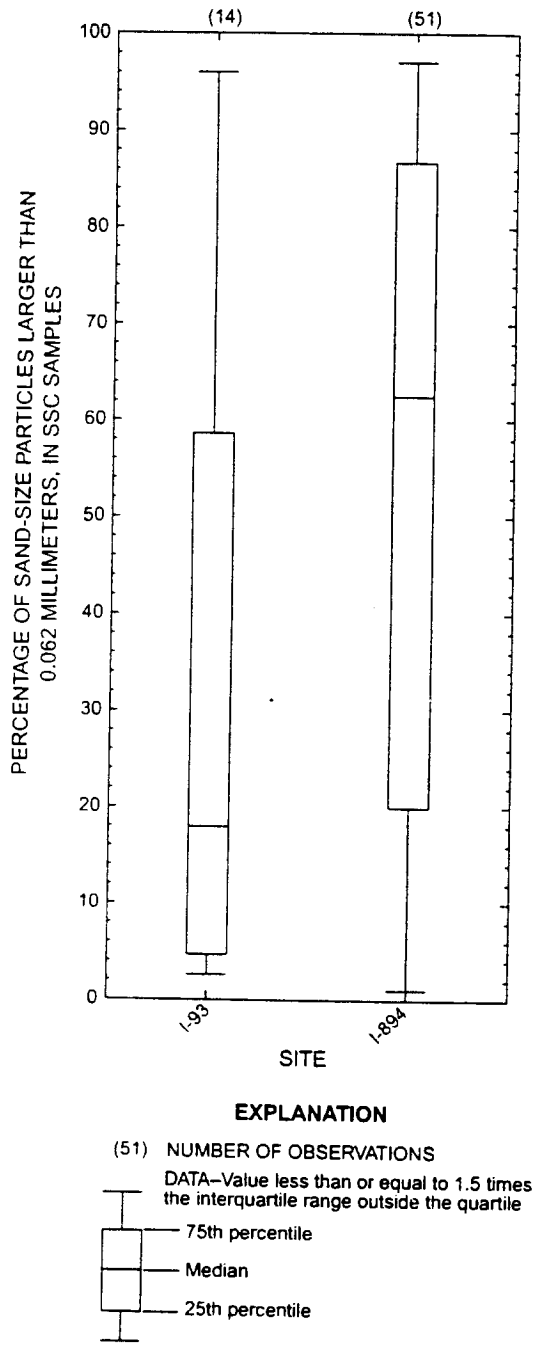

Figure 16. Distribution of the percentage of sand-size particles measured in suspended-sediment concentration (SSC) in highway-runoff samples collected along I-93 in Boston, Massachusetts, and I-894 in Milwaukee, Wisconsin, 1999–2000 (data from Smith, 2000, and R.J. Waschbusch, U.S. Geological Survey, written commun., 2000, respectively).

site-specific conditions (such as the range of concentrations and PSDs) and to implement QA/QC measures to quantify the performance of the sample-analysis methods used.

Perhaps the broadest implication of this systematic problem in the TSS analysis method is for interpretation of the performance of sediment-removal BMPs. For example, figure 17 demonstrates the effect of the analysis method on the calculated removal efficiency of catch basins and oil-grit separators. These devices have a median removal efficiency of about 50 percent when the SSC analysis is used because these BMPs are relatively effective for removing sand-size particles. When the proportion of SSC associated with fine sediments (diameters of less than 0.063 mm) are calculated, this "efficiency" compares well with the TSS efficiency calculated using the TSS analysis (fig. 17). These efficiencies are less than the efficiencies calculated using the SSC analysis because these devices do not effectively retain fine-grained sediments. When the TSS analysis method is used, these artifacts will have several important consequences for the assessment, design, and maintenance of BMPs, including:

- the variability in grain-size distributions for different periods of storm runoff and site to site may confound meaningful analysis of BMP effectiveness;
- the necessary volume of sediment-retention structures may be underdesigned; and
- maintenance schedules for sediment removal from these structures may not be adequate because sedimentation rates may be greater than expected.

These problems may arise if decisions are based on expected TSS capture efficiencies because the TSS values do not reflect the actual sediment retention of larger grain sizes, which are characterized by the SSC method. Problems in analysis, interpretation, and design using the TSS method would be exacerbated in areas where sand is used for winter maintenance.

Particle-Size Distribution

Particle-size distribution (PSD) is the percentage measured by mass, volume, or number of particles in a range of specific sizes, such as those shown in table 4. The American Society for Testing and Materials (ASTM) (American Society for Testing and Materials, 1997a; 1997b) has identified information about the

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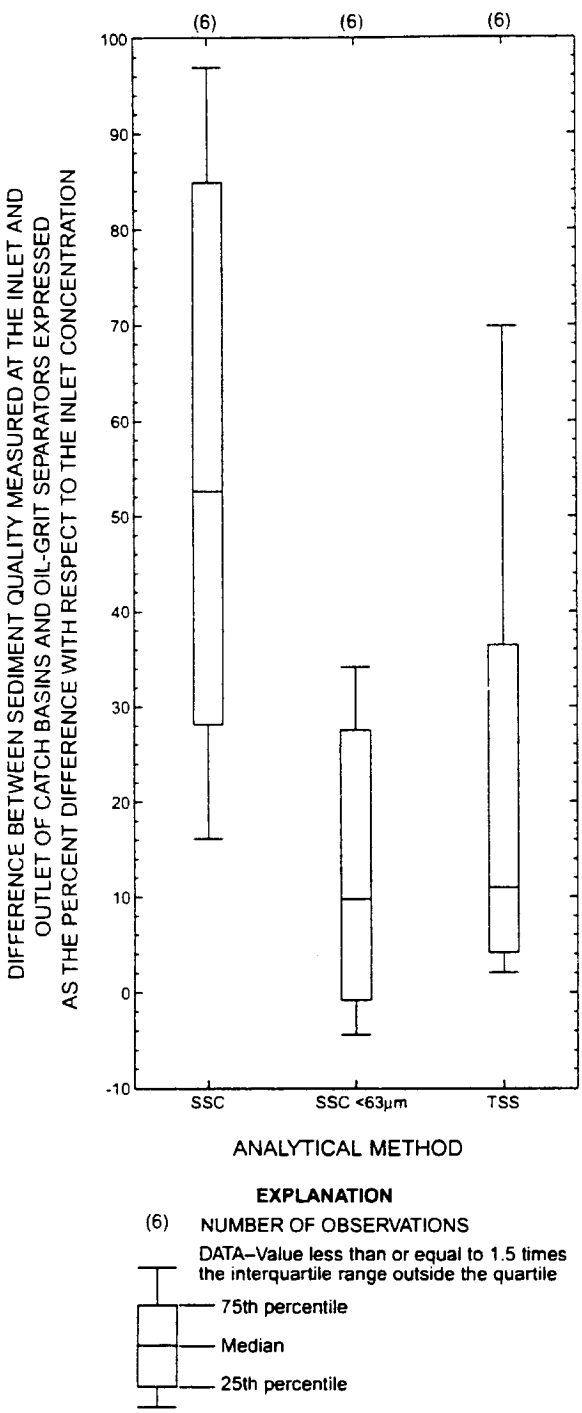

Figure 17. Distribution of differences in sediment measured at the inlet and outlet of catch basins and oil-grit separators in highway-runoff samples collected along I-93 in Boston, Massachusetts (data from Smith, 2000).

PSD to be a necessary component of data sets for environmental sediments. Particle-size analysis is useful for study of the chemistry (Breault and Granato, 2001), transport, and fate of sediment in highway and urban runoff, BMPs, and receiving waters (Kobriger and Geinopolos, 1984). Particle-size analysis may be a measure of sample integrity because the representativeness of individual samples collected may be compared to the grain-size distributions measured for each site. For example, an unusually high sediment concentration may be caused by relatively few sand-size particles in a sample in which the materials at a site are predominantly fine grained, because one medium-size sand grain has the equivalent mass of about 1,000 medium-size silt grains given equal densities. Traditional manual methods used for determining particle size are dry sieve, wet sieve, visual-accumulation (VA) tube, bottom withdrawal (BW) tube, pipet, and microscopy (Guy, 1969; International Standards Organization, 1997a; Percival and Lindsay, 1997; American Society for Testing and Materials, 1999). Electronic methods used for determining particle size include the electrical sensing-zone principal (Coulter Counter), x-ray sedimentation (Sedigraph), laser time of transition (Brinkman Particle Size Analyzer), laser diffraction spectroscopy, and light-optically based image analysis (Matthes and others, 1991; Percival and Lindsay, 1997; Jongedyk, 1999). Each method has different effective size ranges, effective analysis concentrations, and sediment quantity requirements (Guy, 1969; Percival and Lindsay, 1997). For example, sieve analysis has a lower limit of about 0.062 mm, whereas pipet analysis is most effective in the range between 0.002 and 0.062 mm (Guy, 1969; Matthes and others, 1991). Each method is also based on design assumptions that may affect the interpretation of results. For example, grain sizes produced by the pipet and Sedigraph methods are based on the assumption that all sediments in the sample have the specific gravity (and therefore the effective fall velocity) of similar shaped quartz particles (Percival and Lindsay, 1997). It is therefore incumbent upon the investigator to select consistent PSD analysis protocols designed for the concentrations, size range, and other characteristics typical of highway- and urban-runoff sediments. It is also necessary to clearly document the methods used to implement QA/QC measures to quantify the performance of PSD methods (Matthes and others, 1991; Knott and others, 1992; 1993).

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Specific Gravity

Specific gravity is the ratio of the unit weight of the sediment to the unit weight of water at 4°C. It is a unitless measure of density determined by direct measurement of the weight and volume of the sediment sample (Guy, 1969). The ASTM (American Society for Testing and Materials, 1997a) has identified specific gravity as a necessary component of data sets for characterization of environmental sediments. Knowledge of specific gravity of runoff sediments provides information about the settleability of these sediments. For example, Whipple and Hunter (1981) measured substantial differences among the settling rates of different fractions in urban-runoff samples. As previously mentioned, knowledge of the specific gravity may affect interpretation of PSD analysis. It may also provide information about the relative contribution of inorganic and organic components in runoff sediments because the organic fraction is usually less dense than the inorganic (soil) fractions (Butler and others, 1996a). Typically, mineral species (inorganics) have specific gravities that are generally between about 2.5 and 3.5, (Dunn and others, 1980), whereas the organic fraction of stormwater solids are between about 1.1 to 2.5 (Butler and others, 1996a).

Other Sediment Measurements

Other sediment measurements of potential interest for highway and urban runoff and the design and maintenance of structural BMPs include those of settling velocity, the organic content of sediment, particle shape, and specific surface area. The settling velocity is a primary measure that incorporates a number of factors (grain size and shape, and specific gravity) germane to the treatability of solids in runoff. Methods for determination of settling velocity are described by Guy (1969) and Clesceri and others (1998). As presented previously, the organic content of suspended sediments affects the average specific gravity and could thereby affect the interpreted PSD (Guy, 1969). Particle shape represents the aspect ratios of individual sediments and affects settling velocities and PSD measurements (Vanoni, 1975; American Society for Testing and Materials, 1997a). The specific surface area is a function of the shape and texture of the sediments and is an important characteristic of the sediment's ability to act as a carrier of contaminants (Horowitz and Elrick, 1987; Sansalone and others, 1996; Percival and Lindsay, 1997; Sansalone and others, 1998).

DATA-QUALITY CONSIDERATIONS

Data quality, comparability, and utility are important considerations when collecting, processing, and analyzing sediment samples and interpreting sediment data for studies of highway and urban runoff. Results from a sediment study must also be readily transferable from electronic databases, and useful to resource managers and regulators. To meet these objectives, supporting ancillary information must be available that documents the methods and procedures that are used and describes QA/QC procedures that are employed.

Documentation of Methods

Techniques for the collection, processing, and analysis of sediments in highway and urban runoff and in fluvial systems are continually being developed and refined. New techniques and improvements of existing techniques serve to enhance the accuracy and cost-effectiveness of monitoring programs. This evolution of the science, however, makes it increasingly difficult to compare data over time. This is of particular concern to long-term and broad-scale monitoring and assessment programs that draw upon the expertise of a wide range of scientists. The exclusive use of published and proven procedures would help alleviate this concern, but could impede scientific advancement. Studies designed to compare the results of new and existing methods and the practice of thoroughly documenting and describing all techniques employed, however, can help resolve these problems (U.S. Geological Survey, 1991, Intergovernmental Task Force on Monitoring Water Quality, 1995a; 1995b).

Many of the data elements necessary to document methods and procedures used are discussed in Glysson (1989), U.S. Geological Survey (1991), American Society for Testing and Materials (1997a), and Edwards and Glysson (1999), and are include on sediment-station inspection sheets developed by the

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U.S. Geological Survey (1991). Standard sediment-station field inspection forms were designed to record who visited the site, the date and time of each visit, site conditions, the status of equipment and instrumentation, records of instrument calibration, and other information pertinent to the operation of the station that are necessary for data verification (U.S. Geological Survey, 1991). One example of a modified sediment station inspection sheet (Robert Holmes, USGS, written commun., 1994) is provided to illustrate information necessary for documentation of local, regional, and national data sets (fig. 18). The general elements necessary to document the station and field conditions include: station information (linked to detailed location and site characteristics data in the USGS national water data bases), the date and time sediment data are collected, and climatic and hydrologic conditions. Elements necessary to document sample collection methods include the type of sampler, the location in the channel where samples are collected, and the number of and condition of samples collected. When bed-material samples are collected, sampling methods for these samples must be documented as well. Information about the observer is also necessary, especially if sampling is conducted for regulatory programs, which may require chain-of-custody information (U.S. Environmental Protection Agency, 1992; Granato and others, 1998). Information about field quality-assurance steps also are recorded appropriately on the field inspection sheet. Inspection sheets also represent checklists to improve the consistency and comparability of data-collection efforts. For example, if sample water is spilled during transport to the laboratory, field sheet records and practices, such as marking the bottle's water level, may facilitate analysis and interpretation of data. These field sheets should have sufficient space to record remarks describing field conditions such as trash accumulation at the sampling sites, malfunctioning equipment, and other factors that may affect the validity, comparability, or representativeness of samples. These field sheets may be customized to address the data-quality objectives of an individual study and to address site-specific conditions (U.S. Geological Survey, 1991). For example, a water-quality field inspection form designed for use of automated data-collection methods and automatic pumping samplers at a highway-runoff monitoring station is presented in figure 19 (Smith, 2000). All sediment data as well as methods and procedures used in data collection and analysis should be stored in an electronic database(s) with several hard copies stored in alternative locations to avoid the loss of valuable information.

Quality Assurance/Quality Control

In sediment data-collection programs, quality assurance/quality control (QA/QC) efforts are an integral component of all sample collection processing, and analysis operations. Quality-assurance protocols for water-quality data collection have improved substantially since the early 1980s. The guidelines developed by Edwards and Glysson (1999) serve as protocols for sediment data collection by most Federal agencies that collect these data as part of their mission. These guidelines also are consistent with the Wilde and others (1999a) protocols for collection of water samples. Knott and others (1993) provide a quality-assurance plan for field collection, laboratory processing, and office analysis of sediment data. Protocols for collecting stream-water and bed-sediment samples for the National Water-Quality Assessment Program (Shelton, 1994; Shelton and Capel, 1994) are consistent with Edwards and Glysson (1999). Among the protocols relevant to sediment data developed by the International Standards Organization (ISO) are Methods for Measurement of Suspended Sediment (International Standards Organization, 1993), Guidance on Sampling Rivers and Streams (International Standards Organization, 1990b), Guidance on Sampling of Bottom Sediments (International Standards Organization, 1995), and Determination of Turbidity (International Standards Organization, 1990a). Standards relevant to sediment by the ASTM include Terminology for Fluvial Sediment (American Society for Testing and Materials, 1998c), Standard Guide for Sampling Fluvial Sediment in Motion (American Society for Testing and Materials, 1998a), Standard Guide for Core-Sampling Submerged, Unconsolidated Sediment (American Society for Testing and Materials, 1995), Standard Guide for Elements of a Complete Data Set for Non-Cohesive Sediments (American Society for Testing and Materials, 1997a), Standard Guide for the Selection of Maximum Transit Rate Ratios and Depths for the U.S. Series of Isokinetic Suspended-Sediment Samplers (American Society for Testing and Materials, 1998b), Standard Guide for Monitoring Sediment in Watersheds (American Society for Testing and

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U.S. GEOLOGICAL SURVEY WATER RESOURCES DIVISION (Rev. 10-93)

SEDIMENT STATION INSPECTION SHEET

GENERAL

Station #_____Station Name_____
Date_____Party_____
Start Time_____ End Time_____Mean Time_____
Start OSG____End OSG____Mean OSG____Start ISG____End ISG____Mean ISG____
Discharge_____(rating/measure) Remarks_____
Condition of Control_____
Station at LEW_____Station at REW_____
Width_____Mean Depth_____Mean Velocity_____(meas/est.)
Weather_____Water Temp._____Air Temp._____
Stream condition (rise/fall/steady, etc.)_____
Unusual Conditions (surface boils, standing waves, debris, etc.)_____
Remarks_____

SUSPENDED SEDIMENT SAMPLES

SAMPLING METHOD (EWI, EDI, GRAB, Single Vertical) and # of verticals_____
Sampler Type (D-74, DH-48, DH-59, D-77, D-95, DH-95, D-96, Other _____
Nozzle Size (1/4, 3/16, 1/8) _____Mean Vel._____Maximum Transit Rate_____
Samples Cross-section Location: Wading, Cable, Ice, Boat, Upstream side bridge,
Downstream side bridge _____ feet/miles above/below gage
Automatic Sampler Type_____ Number of Samples Collected_____
Condition of Samples_____ Sampling Times in Sync with # of Samples (Y or N)
Condition of Sampler Intake(s)_____
Samples Collected for Determination of Cross-Sectional Coefficient (Y or N)
Duplicate Sample Collected (Y or N)
Remarks_____

BEDLOAD SAMPLES

BED MATERIAL SAMPLES

Time_____GH_____# Verticals_____Sampler (BM54, Other_____)
Location of Sampling Cross Section_____
Sampler working properly (Y or N) Remarks_____
Remarks_____

OBSERVER

Contacted (Y or N) # cases picked up_____ # cases left_____
Observer Sample Inspected On-Site (Y or N) Problems Immediately Addressed (Y or N)
Observer Sampler Inspected for leaks or Need for Adjustment (Y or N)
Remarks_____

QUALITY ASSURANCE

Samplers checked for proper bottle seal (Y or N)
P61/P63 checked for leaks/proper solenoid opening (Y or N)
EDI sample bottles have equal volumes (Y or N)
Maximum transit rate exceeded (Y or N)
Remarks_____

Figure 18. Field inspection sheet to record measurements and stream conditions observed during a visit to a sediment-measurement site (modified from Robert Holmes, U.S. Geological Survey, written commun., 1994).

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U.S. GEOLOGICAL SURVEY, WRD
WATER QUALITY FIELD INSPECTION FORM

Sta. Name:_____ No.:_____
Observations Made By :

Date: _____ Julian Day:_____ Watch Time:_____ EDL Time:_____
EDL Time Reset? No Yes ---> Watch Time:_____ EDL Time:_____
EDL Operational? No Yes Battery Voltage Found :_____Volts
Program Altered? No Yes Battery Replaced? No Yes_____Volts
Down loaded? No Yes File name:_____
Remarks:_____

EDL SENSOR SECTION

Sensor Name	Std. or Field Meas.	EDL Initial @T^0	EDL Serviced @T^0	EDL Adjusted @T^0	Sensor Condition i.e., algae growth, silted
Stage	OG:				
W. T^{0}C					
SC					
SC					
SC					
SC					Cell:
pH					
pH					
pH					Slope:
pH					Offset:
DO					
DO					Press: mm/Hg
DO	ZERO				Salinity corr.:
Turbid	ZERO				Slope:
Turbid					Offset:

Sensor (s) removed from water_____HRMN Returned_____HRMN

Figure 19. Field inspection sheet customized for automatic data collection at a highway-runoff monitoring station (Smith, 2000).

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Sensor(s) Maintenance Comments:_____

Weather: Clear Partly Cloudy Light Medium Heavy Snow Rain Calm Light Breeze

Gusty Wind Very Cold Warm Hot **Snow on Ground?** No Yes____in.

Reference Meter (s)	Make/ Model	Serial No.	Corr. factor applied?
Temperature			None Yes No
Conductivity			None Yes No
pH			None Yes No
Dissolved Oxygen			None Yes No
			None Yes No
			None Yes No

Control:_____

Flow:_____

Automatic sampler(s) log: **C: Complete, NLD: No liquid detect, E: No liquid**

Sampler 1 Configuration:_____ Samples Triggered_____

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24

Remarks:_____

Sampler 2 Configuration:_____ Samples Triggered_____

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24

Remarks:_____

Observations:_____

Figure 19. Field inspection sheet customized for automatic data collection at a highway-runoff monitoring station (Smith, 2000)—*Continued.*

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Materials, 1997b), and Standard Guide for Collection, Storage, Characterization, and Manipulation of Sediments for Toxicological Testing (American Society for Testing and Materials, 1994). QA/QC efforts need to be established at the beginning of a project to ensure that sediment measurements are accurate and representative of the hydrologic system investigated (Guy 1969; Matthes and others 1991; Knott and others 1992 and 1993; and U.S. Geological Survey, 1998a; 1998b; 1999a). QA/QC programs are especially important for all phases of stormwater-flow and water-quality investigations (Clark and Whitfield, 1993; Brown and others, 1995; and Jones, 1999). An effective QA/QC program for sediment data-collection programs would include:

- Frequent and routine site visits by trained/experienced field personnel;
- Redundant methods for measuring precipitation and stormwater flow (Church and others, 1999);
- Technical training for project personnel;
- Frequent review by project personnel of field and laboratory sediment data;
- Quality audits, in the form of periodic internal reviews; and
- Quality audits, in the form of periodic external reviews.

Field instrumentation must be maintained operational and in good working order to ensure the integrity of the data collected, and derivative data must be reviewed on-site or immediately in the office following the site visit. The site must be inspected for debris accumulation, natural corrosion of equipment, vandalism, and other potential problems. Debris can affect measurements by blocking sample-collection intakes and by affecting necessary flow measurements (Church and others, 1999). Frequent maintenance and calibration of equipment and instrumentation is necessary because of the difficult monitoring environment. Field inspection sheets also are part of quality-assurance efforts and these inspection sheets should be archived with project records, and at the least, use of these forms should be mentioned in the QA/QC documentation in project reports (Guy, 1969; Matthes and others, 1991; Knott and others, 1992 and 1993; U.S. Geological Survey, 1998a).

Periodically, it is necessary to do a more detailed review using the entire data record, field notes, and other available information to detect errors or anomalous data. For example, a comparison between flow, turbidity, and measured sediment concentrations for a given runoff period could indicate a bias in one or the other measurement system if the relation for this storm departed from normal values for the site in question. Analysis of field records, including calibration records, adjustments to measured values, and other information, when compared to the data record, may indicate systematic bias, long-term drift, or an abrupt change in the performance of the instrumentation. Quality audits, in the form of periodic internal reviews, are necessary to monitor and implement the project QA/QC program (Jones, 1999). Internal audits establish that the project has a QA/QC plan and that it is being implemented and documented. Also, periodic internal reviews serve as a method to provide technical feedback from subject-matter experts to examine and address problems and (or) potential problems in the data-collection program. Internal reviews should ensure that trained/experienced personnel are available for frequent and routine site visits, that appropriate and robust monitoring systems are in place and collecting data, and that project personnel are examining and interpreting data using appropriate methods on a timely basis. These internal reviews could take place at the proposal stage of the project and then again when the project is about 10-, 40-, and 70-percent complete, or at fixed intervals, such as quarterly or semiannually.

Quality audits, in the form of periodic external reviews, are also necessary to monitor and implement the project QA/QC program (Jones, 1999). External audits should examine project plans, project data, project records, and QA/QC documentation to ensure that study objectives are being met, and to ensure that study objectives will meet the goals of the monitoring project. External reviews should ensure that the project information is properly documented and that the documentation is accessible. Within the USGS, external quality audits include periodic reviews by technical specialists at different levels in the chain of command above the local organizational unit and by technical specialists from discipline offices such as the Office of Surface Water, the Office of Ground Water, the Office of Quality Water, and the Branch of Quality Systems.

To ensure that the sediment data produced or used for highway- and urban-runoff studies are of a known quality and are sufficient to provide long-term comparability and consistency, sediment laboratory quality-assurance programs are needed (U.S. Geological Survey, 1998a; Gordon and Newland, 2000; Gordon and others, 2000). It is therefore necessary to document the name and location of the laboratory, methods used, and the performance of the laboratory in

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one or more quality-assurance programs. The parameters that are typically evaluated as indicators of quality include accuracy, precision, bias, detection limits or performance range, and interference. In analytical chemistry, these performance parameters can be addressed by the use of internal standards and spiked, blind, and blank samples (Jones, 1999).

SUMMARY

This report addresses technical issues pertinent to the methods for the collection, processing, and analysis of water samples for concentrations and physical characteristics of sediment in highway and urban runoff, best management practice (BMP) structures, and receiving waters. The report focuses on sediment-transport issues related to highway use, as opposed to highway construction. Information presented in this report is also applicable for many issues related to sediment in urban runoff from stormwater. Many technical issues associated with sample collection, processing, and analysis must be addressed in order to produce valid (useful for intended purposes), current, and technically defensible data for local, regional, and national information needs. All aspects of sediment data-collection programs need be evaluated and quality-assurance and quality-control (QA/QC) data need be documented so that the comparability and representativeness of data obtained for highway- and urban-runoff studies may be assessed.

The erosive capacity of runoff from highways and urban areas can be substantial because runoff from paved areas, ditches, and storm drains can be hydraulically supercritical and turbulent. The area contributing to surface runoff is usually small, water-surface slope is relatively steep, runoff is concentrated, and surface roughness is low. The mode of transport can be described from the origin of the material as bed-material load and wash load, or operationally (as measured by sediment samplers) as suspended load and bedload. As particle-size distributions (PSDs) increase to include sand-size material, a vertical gradient tends to form with larger particles concentrating nearer the bed. Although some coarser sediment can move as bedload, most highway drainage systems are designed to carry water and maintain sediments in suspension so that the volumetric capacity of the highway conveyance structures is not diminished. Sediment in highway runoff comprises particles derived from atmospheric dust, pavement degradation, vehicle rust, tire degradation, trash, rocks, natural soils, biological materials, or chemical precipitates that are transported by, suspended in, or deposited in flowing water. The rapid response of flow volume and velocity to changes in precipitation of highway and urban runoff drainages complicates the sampling and analysis of sediment in these systems. Therefore, it is necessary to use methods that are suitable to this harsh monitoring environment and to support data collected using QA/QC for these methods.

Representative sample collection for measurement of sediment in highway and urban runoff involves a number of interrelated issues. Temporal and spatial variability in runoff can be large, based on a combination of factors including volume and intensity of precipitation, rate of snowmelt, and features of the drainage basin such as drainage area, slope, infiltration capacity, channel roughness, and storage characteristics. The remoteness or inaccessibility of sites makes it difficult to monitor runoff manually, and it can be difficult to get personnel to the site before the onset of runoff. Costs associated with deployment of trained and properly equipped personnel in addition to uncertainties related to the location and timing of runoff, can be prohibitive for manual sampling of storm-runoff periods. The difficulty in collecting a relatively large number of samples during storm runoff and the dangers to field personnel operating in adverse conditions (including traffic, weather, reduced visibility, and rapid changes in discharge) reduces the practicality of manual sampling efforts. In contrast, automatic samplers can be deployed before and samples can be retrieved after storm runoff, reducing logistics and increasing the safety for field personnel. Automatic pumping samplers typically collect water from the water column by suction and control the sampling rate using the pump speed. Passive sampling devices typically are installed in the flow path and control the sampling rate by placement, orientation, and design of the water intake. Each type of sampler has benefits and design limitations, which must be recognized and quantified to produce representative data. Indirect sediment-measurement methods also may be useful as supplementary and (or) surrogate means for monitoring sediment in runoff. These methods include analysis of available bottom materials, turbidity, and other indirect sediment-measurement methods. All these methods

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have benefits and design limitations, which must be recognized and quantified to produce representative data.

Appropriate sample processing methods are determined by the characteristics of the water sampled and by the analytical and interpretive methods used for data reduction. Water-quality data for highway and urban runoff are generally reported as event-mean concentration EMC to provide summary values that can be used to compare measurements from individual runoff periods at a site or to compare, between sites, measurements from populations of storms. An EMC may be determined by collecting a bulk sample, by physically compositing a number of discrete samples, or by mathematically calculating a flow-weighted composite from analysis of multiple discrete samples taken during the runoff period. Each sample type has certain pre-analysis processing requirements that may affect measured sediment concentrations. Each sample type has certain pre-analysis processing requirements that typically include homogenization and subsampling. Homogenization methods are designed to produce representative subsamples for the analysis process. Subsampling methods are designed to enable different analytical determinations to be made on the subsamples. Processing artifacts can be substantial if the methods used are not appropriate for the concentrations and PSDs present in the samples collected.

Representative analysis of concentrations and physical characteristics of sediment in highway and urban runoff involves a number of complex issues. The two analytical methods most commonly used to determine sediment concentration in a water sample are the suspended-sediment concentration (SSC) method and the total suspended solids (TSS) method. The terms SSC and TSS are often used interchangeably in the literature to describe the concentration of solid-phase material suspended in a water-sediment mixture. The SSC analytical procedure, entails measurement of the entire mass of sediment and the net weight for the entire sample, whereas only a part of the water-sediment mixture (a subsample) is typically analyzed in the TSS method. Although these methods are commonly expected to produce comparable results, recent research indicates systematic differences between the methods. Two studies comparing laboratory analysis results for TSS and SSC found that TSS analysis does not represent the larger grain-size fractions, and therefore, consistently tends to underrepresent the true sediment concentrations. Furthermore, these studies determined that relations between TSS and SSC concentrations are not transferable among sites, even when grain-size distribution information is available. An analysis of data from a coastal and an interior highway in the United States indicates that TSS underrepresents the true sediment concentration, and that relations between TSS and SSC concentrations are not transferable from site to site even when grain-size distribution information is available. TSS data may be fundamentally unreliable and published TSS data may not represent sediment concentrations and loads from highways. When the TSS analysis method is used, these artifacts may have important consequences for the assessment, design, and maintenance of sediment removal BMPs, including consideration that the variability in grain-size distributions from storm to storm and site to site will confound meaningful analysis of BMP effectiveness, that the necessary volume of sediment retention structures may be underdesigned, and that maintenance of these structures may not be adequate because sedimentation rates are greater than expected. These common problems arise if decisions are based on expected TSS capture efficiencies because the TSS values do not reflect the actual sediment retention of larger grain sizes, which are characterized by the SSC method. Further research, however, may be necessary to quantity the scope of this issue in different highway and urban settings.

Data quality, comparability, and utility are important considerations for the collection, processing, and analysis of sediment samples and interpretation of sediment data for highway- and urban-runoff studies. Results from a sediment study must also be readily transferable and useful to resource managers and regulators. To meet these objectives, supporting ancillary information must be available that documents the methods and procedures that are used and describes QA/QC procedures that are employed for highway-runoff studies. Valid, current, and technically defensible protocols for collecting, processing, and analyzing sediment data for the determination of highway-runoff quality therefore need to be documented with study results.

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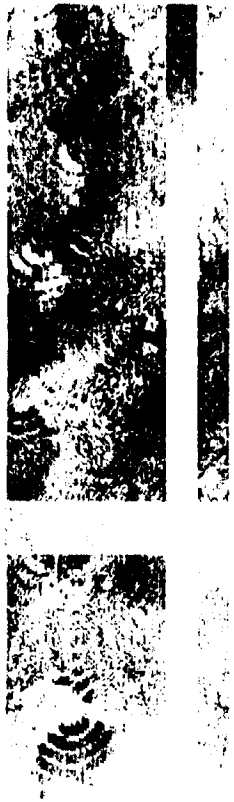

Santa Monica Bay Consortium Catch Basin Insert Study

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in association with Woodward Clyde Consultants and Psomas and Associates

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Background

- Study sponsored by Consortium of Santa Monica Bay area jurisdictions
- Lead agency: City of Santa Monica
- Technical advisory committee (TAC): subset of interested participants

Project objectives

- Evaluate existing and prototype catch basin devices to control pollutants in dry-weather flows and storm water runoff
- Focus on conditions in areas tributary to Santa Monica Bay
- Assist government officials make decisions regarding use of catch basins and inserts

Storm water sampling

- Objective: characterize quality of local runoff in order to generate "recipe" for laboratory testing
- Conducted at 4 locations: commercial and residential land uses during November 1997 through January 1998
- Analyzed samples at UCLA laboratory

Establish performance objectives

- Worked with TAC to define constraints on designs/retrofits
  - may use internal volume or modify catch basin openings if no added flood risk
  - changing top is only allowable modification
  - spend no more than $500 per basin
  - do not increase water depth in front of basin
  - limit cleaning frequency to 2 times/year

Test devices: field

- Dry-weather devices
  - boardovers
  - coarse screens
  - controls
- Wet-weather devices
  - coarse screens
  - basket inserts
  - controls

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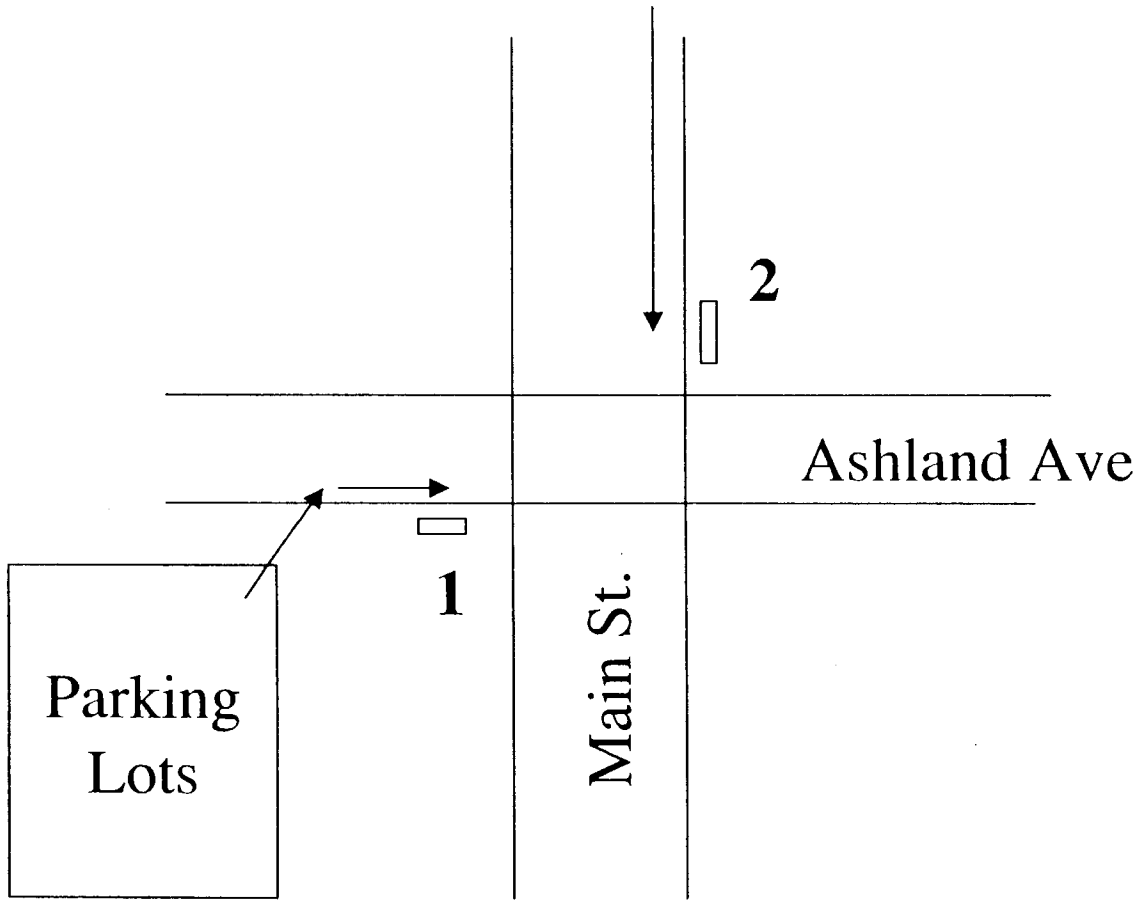
Test devices: laboratory

- Designed new inserts (baffle with settling chamber)
- Evaluated three commercially-available inserts/offline devices
  - Stormwater Management (CSF)
  - AbTech
  - Continuously Deflective Separation (CDS)

Task 2 - Catch Basin Sampling

- Four sampling Locations
  - 1 Vechicular (Ashland Ave and Main St)
  - 2 Light Commercial (Main St.)
  - 3 Residential (Ashland and 4th)
  - 4 Residential (Hill and Lincoln)
- Wide Range of Analyses

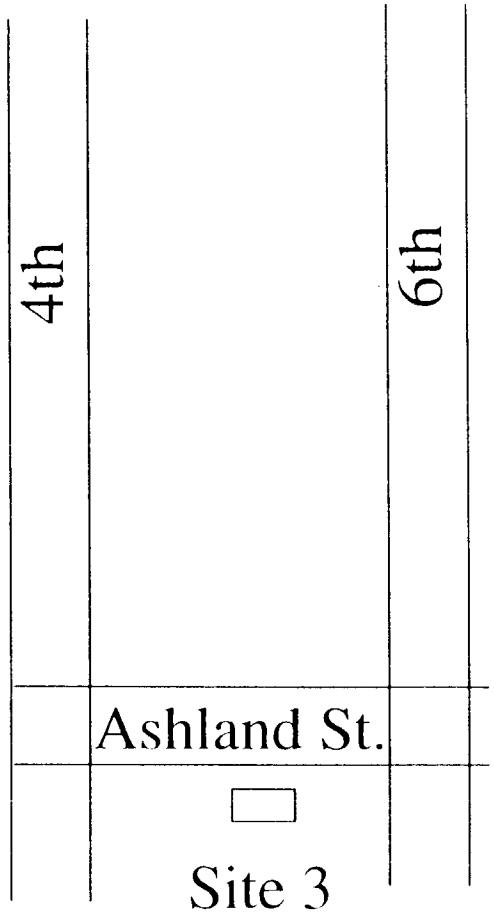
Sites 1 and 2



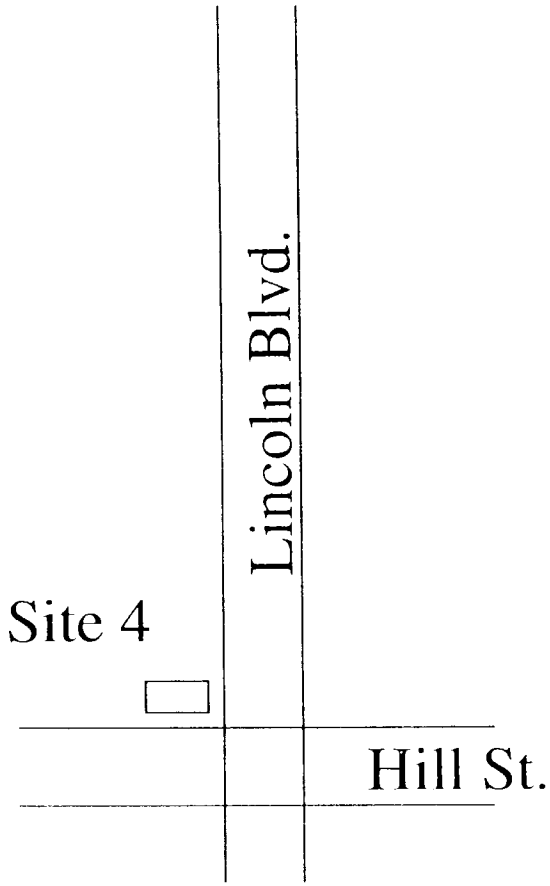
Site 1

Site 2

Site 3



Site 4



Site 4 picture

Sampling Dates

- 9/25/97 0.28 in
- 11/10/97 0.67 in
- 11/13/97 0.48 in (2 sets)
- 11/26/97 0.72 in
- 11/30/97 1.12 in
- 12/5-6/97 3.36 in (3 sets)
- 12/18/97 0.95 in

Average Water Quality

Table 3.5 Water Quality Parameter Means and Standard Deviations of all Data by Site

Water Quality Parameter	Site 1 Mean (mg/L)	Site 1 Standard Dev	Site 2 Mean (mg/L)	Site 2 Standard Dev	Site 3 Mean (mg/L)	Site 3 Standard Dev	Site 4 Mean (mg/L)	Site 4 Standard Dev
TSS (mg/L)	62.0	78.9	42.0	34.0	37.8	35.5	38.3	42.2
VSS (mg/L)	44.3	67.2	23.0	14.2	21.0	19.7	19.9	19.4
Turbidity (NTU)	24.3	26.6	16.9	11.4	13.1	8.3	14.1	10.7
Conductivity (mmho/cm)	178	220	180	177	200	148	171	152
pH	6.5	0.4	6.8	0.3	6.9	0.4	6.9	0.4
Alkalinity (mg/L as $CaCO_3$)	21.2	14.2	24.9	13.6	30.6	17.6	29.0	16.0
Hardness (mg/L as $CaCO_3$)	44.6	46.3	44.2	35.6	47.3	32.5	51.5	44.5
COD (mg/L)	187	227	113	132	113	112	123	127
SPE Oil and Grease (mg/L)	8.2	11.0	5.9	6.1	5.8	5.6	6.4	8.6
Ammonia (mg/L as NH_3-N)	1.5	1.1	1.2	1.1	1.8	1.4	1.0	0.8
Cl^- (mg/L)	30.4	39.9	30.1	31.2	28.8	21.9	23.3	21.0
NO_2^- (mg/L as NO_2-N)	0.18	0.16	0.18	0.16	0.34	0.42	0.19	0.17
DOC (mg/L)	45.2	63.0	37.4	49.3	31.0	31.5	30.8	31.1

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Soluble Metals

Averages (ug/L)

	Site 1	Site 2	Site 3	Site 4		Site 1	Site 2	Site 3	Site 4
Al	89.0	99.1	123.7	165.0	Mn	123.1	55.8	74.2	220.0
As	3.0	1.6	3.3	2.3	Mo	<1	<1	<1	<1
Ba	294.8	178.9	353.7	251.3	Ni	12.6	9.3	16.8	11.6
Be	<1	<1	<1	<1	P	1309	467.4	1572.0	897.7
B	193.9	134.1	375.1	242.3	Sb	<10	<10	<10	<10
Cd	0.8	1.2	0.5	1.0	Se	<10	<10	<10	<10
Cr	2.1	1.3	1.6	2.5	V	4.0	3.5	5.6	4.7
Co	3.1	1.4	1.6	4.9	Zn	783.3	761.1	625.6	391.8
Cu	47.7	39.7	47.4	35.2	Ca	12.5	8.5	15.7	19.6
Fe	536.9	154.8	243.0	197.9	Mg	5.2	2.7	5.5	4.7
La	<5	<5	<5	<5	K	8.7	4.1	12.6	6.4
Pb	3.2	2.7	3.9	9.1	Na	34.7	19.7	28.5	24.5
					Si	1.1	0.9	1.9	2.6

Large Solids

Sieve Analysis of Suspended Solids from Site 1

Date	Rainfall (in)	TSS (mg/L) Mesh Size > 100	100/200	Mesh Size 200/325	< 325	Total
1/4/98	0.42	6.41	3	2.87	12.3	24.6
1/19/98	0.25	3.71	2.47	2.16	8.3	16.6
2/14/98	2.53	5.26	1.92	1.62	8.8	17.6
Average		5.13	2.46	2.22	9.8	19.6
Percentage		26	13	11	50	100

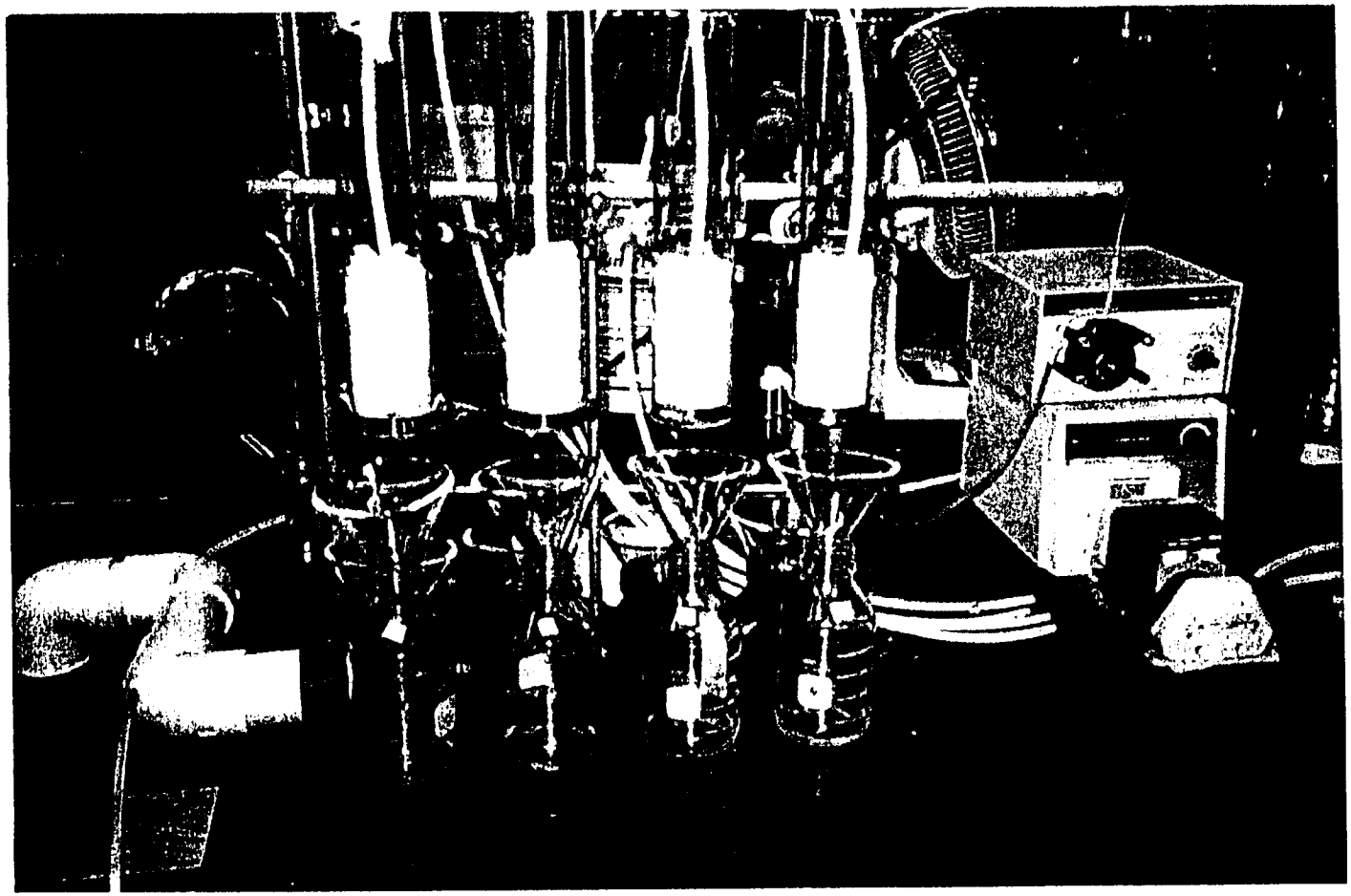
Sampling Conclusions

- Land Use
- Soluble O&G 5 ~ 8 mg/L
- Modest trend towards first flush
- 39% solids greater than 200 microns
- Suggests:
  - O&G concentrations much less than suggested by some sorber vendors
  - Sedimentation promising

Insert Testing

- Lab Testing
  - Bench top
    - sorbers
  - Full Scale
    - sedimentation
    - sorbers
- Field Testing
  - Wet and Dry Weather
  - Screens, Boards and Baskets

Bench Top Testing



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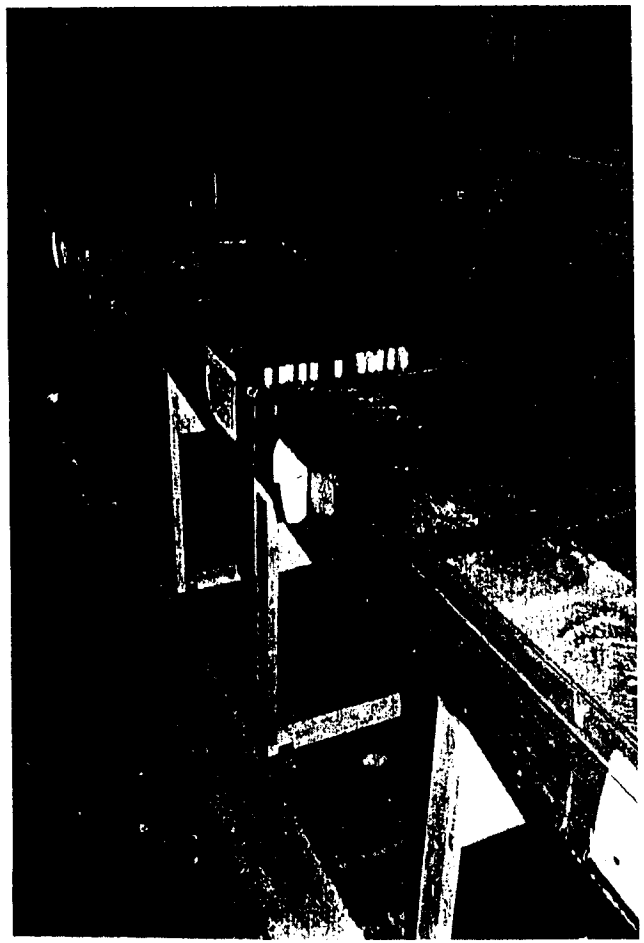

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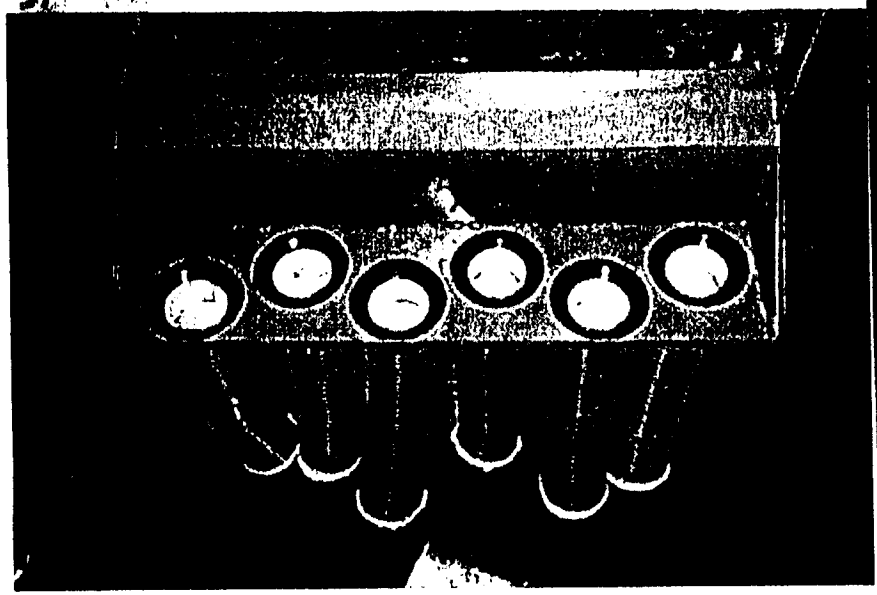

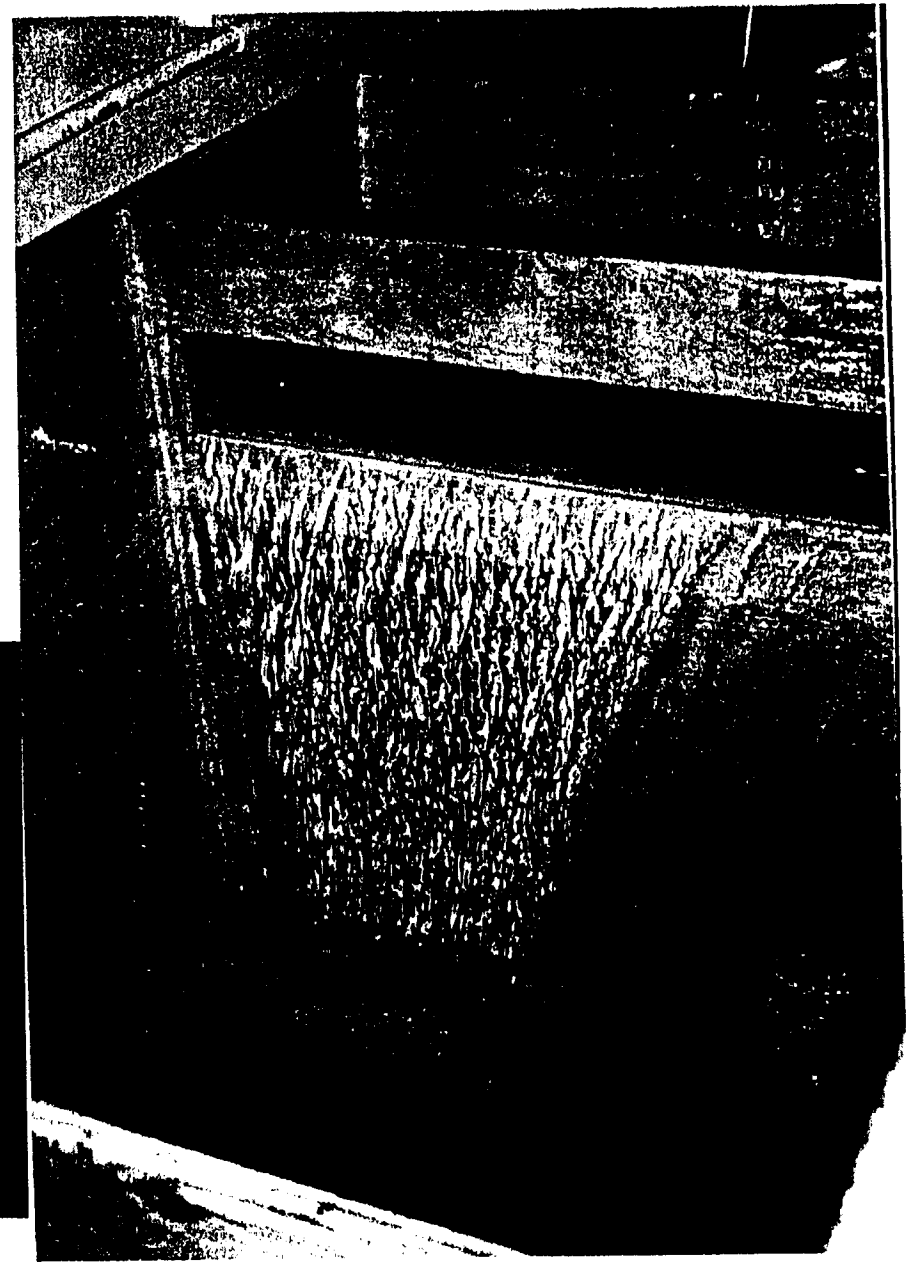

Sedimentation

Sand Removal Efficiencies (percent) by weight

Sand Size (mesh)	Test 1 (50 GPM) 1.6 min HRT 19060 gal/ft2day	Test 2 (100 GPM) 0.8 min HRT 38120 gal/ft2day	Test 2 (150 GPM) 0.53 min HRT 57180 gal/ft2day
30-60	73	72	59
60-100	63	45	42
100-140	60	37	30

CDS Device

CDS Removal Efficiency (percent)			
Sand Size	Fine Screen	Medium	Coarse
> 40	59	59	59
40/60	97	78	58
60/100	69	47	41
100/200	21	20	19
Total	61	51	44

by [illegible] note

Suitable for Tech.

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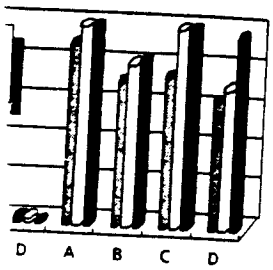
Conclusions

- Inserts possible - but still have work to do.
- More field scale testing required
- Full scale, field testing to demonstrate flood protection
- Inserts can remove up to 70 to 80% of free O&G, but we don't know for how long
- TSS removal in some cases from 40 to 60%



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Catch basin inserts to reduce pollution from stormwater

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Abstract Stormwater contamination represents the largest source of contaminants to many receiving waters in the United States, such as Santa Monica Bay in Los Angeles, California. Point sources to these same waters generally receive secondary or better treatment before they are released, and they are usually discharged through outfalls that diffuse the wastewater plume to prevent it from contacting the shoreline. Stormwaters receive no treatment and reach the receiving waters through a variety of ways, but most enter through catch basins or inserts to storm drains that terminate at the beach or in shallow coastal areas. Under these conditions, the stormwater discharge may have greater impact on the quality and utility of the receiving water than the treated wastewater discharges. One method of reducing pollution is to equip catch basins with an insert that can capture pollutants. A number of commercially available devices exist but few have been evaluated by independent parties in full-scale applications. A series of tests using bench and full-scale devices under both laboratory and field conditions were conducted to evaluate their ability to remove trash and debris, suspended solids and oil and grease in stormwaters. The results presented in the paper should provide a basis for future insert development and application.

Keywords Best management practice; catch basins; litter; stormwater; urban runoff

Water Science and Technology **Vol 44 No 7 pp 23–34** © 2001 IWA Publishing and the authors

Introduction

Most industries and municipalities in the United States have full secondary wastewater treatment, and some have nutrient removal and filtration. As a consequence of these reductions in water pollution, stormwater now represents the greatest threat to aquatic habitants in the United States. Stormwater quality has been largely ignored in many areas, although there is usually concern for flood control and flood damage prevention. As a result, we have stormwater management systems that prevent floods at the expense of environmental protection.

Los Angeles is a good example of an area that has emphasized flood control at the expense of environmental protection. In this highly urbanized area there is little opportunity to reduce stormwater pollution through traditional means. The average imperviousness is more than 60% in many cases. Land values are such that it is prohibitively expensive to retro-fit storage basins or infiltration zones. This paper addresses a potential best management practice for such urbanized areas. The stormwater system has been constructed with catch basins, which may be several cubic meters in volume. These catch basins can be retrofit with devices, called "inserts", to capture pollutants. A number of commercially available devices exist, but few have been evaluated by independent parties in full-scale applications. The authors conducted a series of tests using bench and full-scale devices to remove trash and debris, suspended solids (TSS) and oil and grease (O&G). Field tests were also performed with boards, screens and baskets to observe their ability to remove or prevent debris from entering storm drains. The results are sufficiently promising to suggest additional testing with a variety of devices.

Background

Santa Monica Bay is the receiving water for a major portion of the City of Los Angeles

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metropolitan area. The watershed is 1072 km^2, and is largely urbanized, serving a proportion of the three million people in Los Angeles and more than 11 million people in the metropolitan area. Only two wastewater treatment plants discharge directly into the bay; the largest is the Hyperion Treatment Plant (~1.3×10^6 m^3/day). This plant has recently achieved full secondary treatment, and discharges secondary treated wastewater via an 11 km outfall. The second source is a petroleum refinery that has advanced wastewater treatment. Another source is Los Angeles County's Joint Water Pollution Control Plant (~1.3×10^6 m^3/day, ~60% secondary), which discharges outside of the bay, and is upgrading to secondary treatment. Currents carry the partially treated wastewater into the bay.

The improved treatment has decreased pollutant discharge to the bay by more than an order of magnitude during the past 20 years. As a result, non-point sources now contribute an increased fraction of the total pollutant mass to the Bay (Wong *et al.*, 1997). The non-point contribution is already the major source for many pollutants, e.g. heavy metals, and will become the major source for many more pollutants as full secondary treatment is achieved. Reclamation and water conservation will further reduce point source contamination to the bay.

Various agencies, cities and environmental advocacy groups have proposed structural methods for reducing stormwater pollution. These methods are all difficult to employ because they are small-scale solutions that must be applied to a very broad area, across many jurisdictions with varying interests in controlling stormwater pollution. One proposed method for controlling discharges is to use catch basin inserts.

Catch basin inserts are devices that can be placed into a catch basin or stormwater insert, which will in some way reduce pollutant discharge to the receiving water. A variety of devices have been proposed and marketed, but very few have been evaluated by independent sources, or have been used long enough to create a record of performance. In order to establish creditable performance of insert devices, a consortium composed of the Santa Monica Bay Restoration Project and 14 other Santa Monica area jurisdictions funded a two-year study to determine if inserts are a viable method for controlling stormwater pollution. The results of this initial study (WCC, 1998) were sufficiently promising to warrant additional laboratory testing and a field study.

Objectives were established for testing and insert development. These were based in part upon environmental impact of the pollutants, but in greater part upon the ability of a hypothetical device to remove the pollutant in the constrained volume of a catch basin (generally only a few cubic meters). Litter (trash, debris, etc.), particulates and oil and grease were selected as pollutants of concern. Litter was selected because of its interest to regulators and its high visibility with the public. Total Daily Maximum Discharge Limits (TMDLs) will soon be applied to the Santa Monica Bay Watershed, and litter will be among the first. Particulates, as measured by total suspended solids (TSS) are especially important because a large fraction of the heavy metals in stormwater are adsorbed to their surfaces. Oil and grease, especially oil and grease from vehicular areas, is important because it may contain many anthropogenic compounds that may be toxic to aquatic life.

The approach was divided into two parts: dry and wet weather. This was required because of the seasonal rainfall and the desire to collect litter during the long dry period (generally April to November). It was envisioned that controls would be used in dry weather that would be removed in the wet season. Additionally, public agencies were adamant not to increase flood risks. The approximate cost of installation should be no more than US$ 500; cleaning should be infrequently required. A survey of the member cities suggested that, on average, catch basin cleaning occurred no more frequently than once every two months for beach communities, and approximately once per year for Los Angeles County, as a whole. A problem-solving, practical approach was required. The inserts should not

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increase flood risk and should only marginally change the way stormwater is removed from streets, without increasing the accumulation on streets. Safety considerations such as avoiding confined space entries were important. The public agencies responsible for managing the inserts would soon tire of them if they could not be conveniently, economically and safely maintained.

A sampling program was conducted and differed from previous programs in that samples were collected directly from stormwater on street surfaces, just prior to entry into catch basins. Litter was not measured in the water quality program but was measured during the dry periods as accumulation in the catch basins.

Sampling program

Four locations were selected and sampled during the storm events of the 1997–1998 wet season. This was significant in that it is an El Nino year, and rainfall was at least 200% greater than normal. Table 1 shows the sites and information about them. They were all in the City of Santa Monica and within 4 km of each other.

Samples were taken by scooping 100 to 200 ml at a time until 8 l samples were collected. For short storms only one such sample was collected. For longer storms, three samples were collected and averaged. The oil and grease concentrations were measured by solid phase extraction (Lau and Stenstrom, 1995) and do not include the oil adsorbed to suspended solids. Table 2 shows the mean and standard deviation of conventional water quality parameters for 14 storm events between October 1997 and February 1998. Generally, water quality is worse for Site 1, although the variability tends to make statistical significance

Table 1 Site description

Site number	Land use type	Area (m^2)
1	Commercial (parking lot)	14,000
2	Commercial (streets with small businesses, shops, restaurants, etc.)	7,000
3	Single and multifamily residential	23,000
4	Single and multifamily residential	18,000

Table 2 Stormwater quality (mean followed by standard deviation)

	Concentration							
	Site 1		Site 2		Site 3		Site 4	
Water quality parameter	Average	Std. dev.	Average	Std. dev.	Average	Std. dev.	Average	Std. dev.
TSS (mg/l)	55.1	71.6	38.6	32.3	32.7	33.0	34.1	38.2
VSS (mg/l)	38.5	60.5	21.6	14.7	18.5	18.2	18.1	17.7
Turbidity (NTU)	21.2	24.4	14.4	11.3	11.4	8.2	12.0	10.4
Conductivity (mmho/cm)	153.3	199.4	155.2	163.3	180.3	144.2	151.4	146.0
pH	6.4	0.4	6.7	0.4	6.8	0.5	6.9	0.6
Alkalinity (mg/l as $CaCO_3$)	19.1	13.2	22.5	13.0	27.8	16.7	26.0	15.6
Hardness (mg/l as $CaCO_3$)	38.8	42.4	37.8	33.8	41.3	31.1	44.9	41.2
COD (mg/l)	171.7	205.0	100.9	119.3	106.0	102.5	111.3	116.3
SPE oil and grease (mg/l)	7.4	10.3	5.5	5.7	5.3	5.2	5.8	8.0
Ammonia (mg/l as NH_3–N)	1.3	1.1	1.0	1.0	1.6	1.3	0.9	0.8
Cl^- (mg/l)	26.6	36.0	25.6	28.8	24.7	20.9	20.7	19.2
NO_3^- (mg/l as NO_3–N)	0.1	0.2	0.1	0.1	0.3	0.4	0.2	0.2
DOC (mg/l)	40.1	57.1	31.4	44.9	26.8	29.1	26.3	28.8

Av.=average; Std. dev.=standard deviation

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Table 3 Selected total metals and percent adsorbed to suspended solids

Metal	Concentration							
	Site 1		Site 2		Site 3		Site 4	
	µg/l	%	µg/l	%	µg/l	%	µg/l	%
Aluminium	2235	96	1141	91	1335	91	678	76
Copper	103	53	42	6	52	8	40	11
Lead	45	93	4	33	7	46	11	17
Nickel	75	83	24	61	38	56	39	71
Zinc	2601	70	2062	63	2377	74	1321	70

%=percentage of particulate phase

Table 4 Size fraction of TSS from site 1

Size distribution (µm)	Distribution (%)
> 150	26
150 – 75	13
75 – 45	11
< 45	50

testing difficult. Trash and debris were not quantified, but trash and debris from the commercial sites was obviously greater. Table 3 shows the results for selected metals (only four storm events), as a total concentration and the distribution that was adsorbed onto the suspended solids. These results tended to confirm that metals were associated with the suspended solids.

Toward the end of the sampling period, various insert devices had been evaluated, and it became apparent that the devices could remove larger particles. Therefore additional sampling was performed to determine the size of the particles that compose the TSS. Site 1 was monitored for three storms and the TSS was determined by bailing several hundred litres of water through sieves. Particle sizes are shown in Table 4. These results suggest, for example, that a device that could remove particles larger than 75 µm could remove 39% of the TSS.

Insert evaluation

A survey of all commercially available inserts was performed. At the time of the survey (1997–1998), no devices were found that met all the criteria. A number of promising technologies were found that could treat stormwater, but not for the most common catch basin geometry used in greater Los Angeles. After some review, a concept was developed for a basket that could be inserted and removed through the opening of the catch basin, as shown in Figure 1. Several manufacturers offered prototypes featuring this general concept. This device has the advantage of being useful for both dry and wet weather applications. This design has the advantage of easy installation. An insert that is flexible, or is no greater in width than the opening in the curb, can be inserted and removed from the street. Two chains or cables to the curb support the insert. Workers do not need to enter the catch basin, which in some places is considered a confined space. Alternatively, if worker entry to the catch basin is permissible, the inserts can be installed by bolting to the interior wall. Additionally, high flows are directed around the insert, and flood risk is not increased. Additional material including photographs is available elsewhere (WCC, 1998).

The climate in Southern California presents a special opportunity for dry weather control. The litter that accumulates during the spring and summer, if not removed from catch

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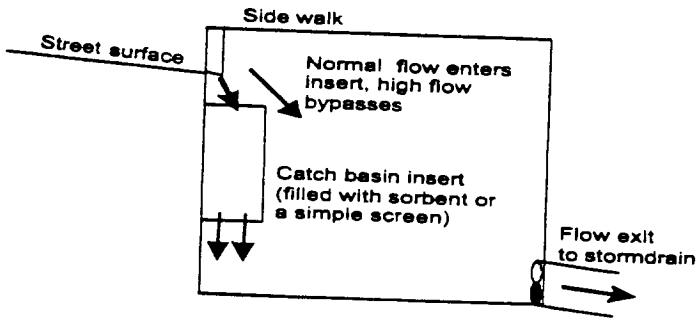

Figure 1 Elevation view of the model catch basin insert developed in this study. Typical minimum basin dimensions are 1 m tall by 0.75 m deep by 1 m wide. The minimum opening is typically 0.15 m

basins, is swept into the bay by the first large storm of the season. To mitigate this problem, the basins are cleaned in September or October. One community has routinely covered catch basins (curbside inlet only) in the dry season to prevent litter build up, insect and rodent problems. Street sweepers then remove the litter, and street sweeping is routinely practiced in these locations. The cover consisted of a plywood board, extending the entire length of catch basin with a gap of 1–2 cm between the bottom of the board and the pavement to allow for nuisance water to enter the basin. The covers or "boardovers" are used only for catch basins in sensitive or high litter-producing areas, and must be removed prior to the rainy season.

To better understand the utility of this practice, two catch basins were covered with plywood and two with wire screens with 2.5 cm square openings. Trash accumulation was monitored. The screens and boards provided roughly equal performance, preventing more than 95% of the build-up in the catch basin, as compared to controls with no covers. Tests were conducted with conventional street sweepers to show that they were capable of removing material that accumulated at the bottom of the covers, and that the sweeper did not destroy the covers. The covers are especially useful in areas with high pedestrian traffic.

Tests to evaluate the inserts' ability to remove contaminants from flowing stormwater were conducted in phases at different scales. Bench scale tests, full-scale laboratory tests and field tests were conducted. Field tests were conducted primarily during the second year of the study. The majority of the testing evaluated oil and grease removal. Many commercially available inserts or stormwater treatment devices claimed that sorbents could be used to remove the oil and grease from stormwater. Previous tests by the authors (Lau and Stenstrom, 1995) also suggested that this might be promising.

Tests were first conducted in columns with 5 cm diameter and height of 5 cm, with mixtures of used motor oil (to simulate the oil and grease in stormwater from commercial areas) and tap water using many different types of sorbents. The oil and grease concentration was generally set to approximately 25 mg/l, which is higher than found in this study, but closer to concentrations of oil and grease found in earlier studies by the author (Stenstrom *et al.*, 1984; Fam *et al.*,1987). Emulsified oil was produced by intensely blending used motor oil with 1 l of tap water to produce a "stock" mixture, which was then further diluted when pumped to the column. Free oil and grease was produced by pumping oil and grease using a syringe pump into a mixing "tee" which was then applied to the columns. The combined flow was allowed to "trickle" through the loosely packed column.

Table 5 shows some of the results. The reported efficiencies are for the period when the sorbent remains "fresh" or unexhausted. As the sorbent is saturated, its efficiency will decline. The mass of adsorbed material per unit mass of sorbent, analogous to "Q" or "M" for activated carbon isotherms, is an important parameter for overall operation. It

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Table 5 Removal efficiencies of various sorbents

Sorbent type	Oil and grease type	Removal efficiency (%)
OARS polymer	Emulsified	3
Activated carbon	Emulsified	11
Aluminium silicate (e.g., perlite, Xsorb)	Emulsified	~0
Straw	Emulsified	~0
Compost	Emulsified	~0
OARS polymer	Free	88, 91
Aluminium silicate (e.g., perlite, Xsorb)	Free	88, 91, 94, 89
Compost	Free	28, 49
Polypropylene (type 1)	Free	86, 92
Polypropylene (type 2)	Free	78, 85

Table 6 Summary of OARS insert device tests

Test no.	Prototype no.	Sorbent condition	Q (l/min)	Influent O&G conc. (mg/L)	Removal efficiency (%)	Final M** (g)
A	1	New	56	20.7	91	11
B	2	New	56	14.1	74	6
1	2	Used in the field*	56	8.4	73	40
2	2	Used from test 1	56	24.7	79	172
3	2	Used from test 2	132	10.7	62	275
4	3	New	132	19.0	78	233
5	3	Used from test 4	132	14.0	65	374
6	3	Used from test 5	132	10.9	46	452
				Inf. TSS (mg/l)		Mesh size
8	3	From test 6		66	99	40
				66	96	60
				66	78	100
				200	91	Average
				PAHs (nominal conc. 50 µg/l)		
9	3	New		Acenapthene	34	
				Fluorene	31	
				Phenanthrene	33	
				Anthracene	61	
				Fluoranthene	33	
				Pyrene	42	
				Chrysene	26	
				Benzo(a)pyrene	16	

* does not include oil and grease removed in the field;
** M = total mass of O&G absorbed (g)

determines the sorbent replacement frequency and therefore the economics of operation. Further work in our laboratory is ongoing to determine these parameters. The sorbents shown in Table 5 are similar, or very similar, to commercially marketed products. The polypropylene materials are used in oil spill control pads and booms. The straw is also used for oil spill clean-up.

None of the sorbents was effective in removing the emulsified oil and grease in this type of experiment. The polypropylene sorbents were evaluated in other tests with 8 to 12 hour contact times and were able to remove 40% to 60% of the oil and grease. If tightly packed

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into columns, they will remove emulsified oil and grease from waters pumped through under high pressure, but this filtration procedure is not economically feasible for stormwater.

A new series of tests was performed in the full-scale catch basin simulator. This simulator is composed of a stilling chamber, a 0.6 m wide flume that simulates street surface, and a catch basin with a 0.9 m wide opening. Contaminants are released into the flume at controlled rates to produce the desired concentrations. Tap water is used for stormwater. This size is the same as the smallest catch basin routinely constructed by the Los Angeles County Department of Public Works. It was constructed of plywood and cement and built above grade to allow easy access. The 0.9 m opening could accommodate a variety of types of inserts. The inserts were temporarily clamped to the walls of the catchbasin and were easily changed and refitted, as needed.

Two prototype designs were extensively tested. The first used OARS sorbent, which was placed in metal boxes with open tops and screened bottoms. Stormwater flows from the top, through the OARS sorbent, which has a particle size from 5–30 mm with a density of 0.22 g/ml (our measurements, not the manufacturer's specifications). The internal arrangement of the box traps suspended solids and trash. This allows the box to perform as oil and grease, suspended solids, and trash removal device. It also means that in installations where high trash and suspended solids are present, the box may clog before the oil sorption capacity is reached. The second insert extensively tested used polypropylene cloth as a sorption/filtration media. The cloth is supported by a geotextile used for stabilizing soils. The cloth is available in different weights. The geotextile has openings of approximately 1 cm by 8 cm. The prototype inserts have a metal collar at the top, which forms the support for the geotextile. The insert is flexible and can be compressed for insertion though an opening smaller than its height. This design has all the previously cited advantages, and can also be easily constructed in custom sizes.

Tables 6 and 7 show the results for both sorbents. The oil and grease removal efficiency ranged from 40% to more than 90%, depending upon sorbent condition and influent concentration. Removal efficiency was generally higher with higher influent concentrations. The media used in tests 1 and 2 for OARS had been used in the field for four months and represented partially used sorbent. Several tests (Figures 2 and 3) were conducted using the same media, in an attempt to exhaust the media.

Also shown in Tables 6 and 7 are test results for TSS and PAH removal. For the case of TSS, sand particles were sieved and recombined to produce an evenly divided mixture, by mass of sand with US standard meshes of 40, 60, and 100 (approximately 400 to 120 μm). The box removed 99% of the large particles and 78% of the smallest particles. PAH removal was measured by spiking tap water with known masses of PAHs and then measuring effluent concentrations. The removal efficiency ranged from 16% to 61%. Again, the total capacity of the insert was not determined, so the mass of solids or PAHs that can be removed before maintenance is not known. This is the subject of further testing in our laboratory, and should be evaluated in the field as well.

Field tests

Field tests were conducted in the second year of the project at commercial and residential sites. Six sites were initially selected. Three used the polypropylene style insert (two in commercial areas) with double thickness liners, two used the OARS containing insert (one in a commercial area), and one used a simple wire mesh basket (~1 cm opening, in a residential area) with no sorbent or filter media. The inserts were observed to bypass flow at the greatest runoff condition and gradually bypassed more flow as they became clogged. After about two months of active rainfall, the bypassing became more frequent and the polypropylene sorbents were replaced with medium screens (see test 14 in Table 7).

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Table 7 Summary of a polypropylene insert device tests

Test no.	Liner type	Sorbent condition	Q (l/min)	Influent O&G conc. (mg/l)	Removal efficiency (%)	Final M* (g)
1	12 oz	New	473	13.5	65	121
2	12 oz	New	283	28.8	82	200
3	12 oz	New	56	37.0	86	54
4	12 oz	New	720	12.7	53	145
5	12 oz	used from test no. 2	283	26.3	78	569
6	12 oz	used from test no. 5	283	21.4	79	714
7	12 oz	used from test no. 6	283	30.2	70	1400
8	12 oz	used from test no. 7	283	23.9	58	2058
9	12 oz	New	283	8.1	56	157
10	12 oz	New	283	17.6	63	366
11	12 oz	New	283	30.5	59	578
12	8 oz	New	283	8.1	49	133
13	Double bag	New	283	11.0	74	274
			TSS (mg/L)			Mesh size
14	Screen	New	283	66	34	40
				66	2	60
				66	0	100
				200	12	Average
15	12 oz	New	283	66	98	40
				66	96	60
				66	95	100
				200	96	Average
				PAHs (50 ug/l)		
16	Double bag	used from test 13		Acenapthene	55	
				Fluorene	51	
				Phenanthrene	58	
				Anthracene	88	
				Fluoranthene	61	
				Pyrene	56	
				Chrysene	82	
				Benzo(a)pyrene	69	

*M = total mass of O&G absorbed (g)

Testing ended for the OARS type sorbents. When stormwater bypassed the insert, there was no change in street runoff rate or increased accumulation on the street surface; the clogged insert had no impact on stormwater removal rate from the street. Sampling was performed as before, except that effluent samples were also collected.

Each residential site was ~12,000 m^2 in area, and the three commercial sites had areas ~5000 m^2 each. Table 8 shows the average water quality for the second year of the study. The values are similar to those shown in Table 2. The standard deviations are high, which is typical for stormwater. Site 2 in Table 2 is similar to the commercial sites used in the second year. The residential sites in the two studies are similar in land use and housing density. The high standard deviations mask water quality comparisons; however, turbidity, COD, DOC, chloride, SPE oil and grease and are higher in the commercial sites (one-tailed test at $\alpha = 0.15$).

The water quality data shown in Table 8 serves as the influent for an efficiency test of the inserts. Effluent samples were collected from the insert using a cup on a stick. Samples were collected when the inserts were not bypassing. Removals for the polypropylene insert

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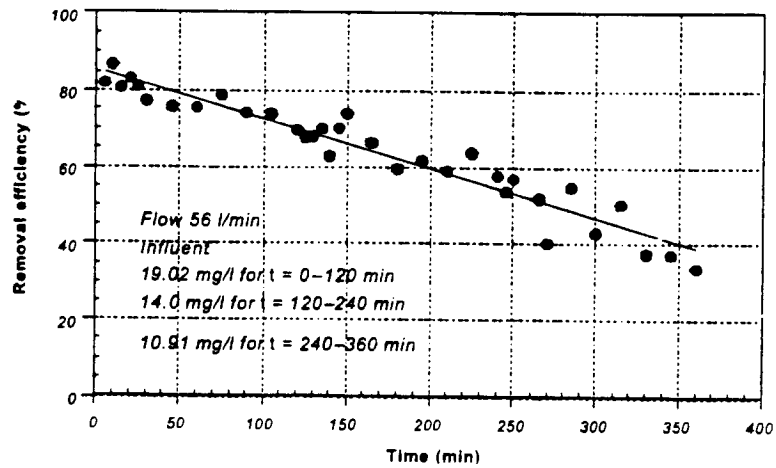

Figure 2 Oil and grease removal efficiency versus time for an insert using OARS sorbent

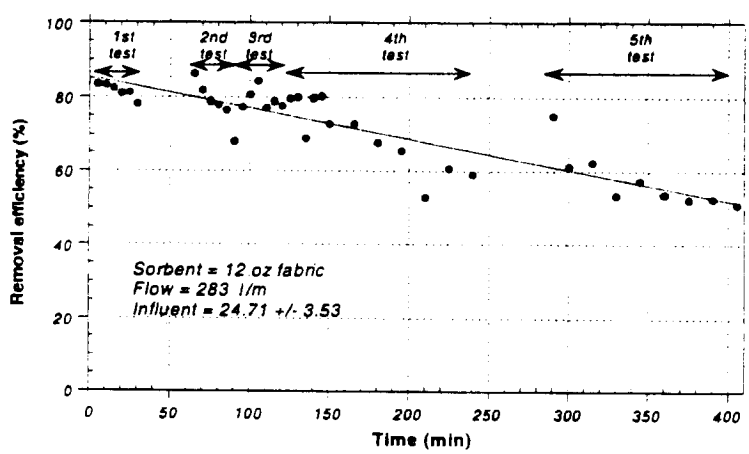

Figure 3 Oil and grease removal efficiency versus time for an insert using polypropylene sorbent

averaged 21, 36 and 34% for TSS, VSS and turbidity, respectively. The OARS device averaged 21, 9 and 12% for the same parameters. The variability in oil and grease removal rates precludes making any conclusion. Table 4 suggested that 26% of the sediment in stormwater might be removed by a filter that captures solids greater than 150 µm. The removals in actual field test are below this prediction, but are not too much different, especially considering the highly variable nature of stormwater. The TSS procedure captures 100% of all particles greater than 0.8 µm; the majority of the material that composes suspended solids is less than the size that can be removed by insert filters.

At the end of the study, the polypropylene bags and screens were removed and the contents were air dried. The material smaller than 12,700 µm (0.5 in) was weighted, screened and reweighed. Table 9 shows the results from the first part of the study. The inserts at the two commercial sites tended to recover smaller particles. Table 10 shows the results for the second part of the study. This study used a much coarser mesh screen, but still recovered many small particles. Again, there is much more finer material at the commercial sites.

The final data reduction was to calculate an equivalent concentration of captured material per unit of runoff volume. This is similar to an event mean concentration, in that the total runoff volume can be multiplied by the coefficients to produce an expected mass of

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Table 8 Water quality parameters for the second year. Number of observations = 16 for commercial sites and 14 for residential sites

Water quality parameter	Commercial		Residential	
	Mean	Std. dev.	Mean	Std. dev.
TSS (mg/l)	54.9	41.7	43.2	39.4
VSS (mg/l)	23.5	18.4	20.0	15.7
Turbidity (NTU)	32.5	23.7	15.6	10.0
Conductivity (mmho/cm)	136.5	95.1	118.8	61.8
pH	6.9	1.1	7.1	0.8
Alkalinity (mg/l as $CaCO_3$)	27.4	22.0	28.7	16.7
Hardness (mg/l as $CaCO_3$)	37.9	29.5	35.9	17.5
COD (mg/l)	147.6	113.5	103.6	66.7
DOC (mg/l)	36.4	33.0	22.9	11.5
SPE Oil and Grease (mg/l)	16.6	21.7	5.4	3.5
Ammonia (mg/l as NH_3–N)	1.1	2.1	0.5	0.6
Cl^- (mg/l)	13.7	10.4	7.2	6.0
NO_2^- (mg/l as NO_2–N)	0.1	0.1	0.1	0.0
NO_3^- (mg/l as NO_3–N)	0.7	0.6	0.7	0.4
SO_4^{2-} (mg/l)	9.3	9.6	7.3	4.7

Table 9 Sieve results for the first part of the study

Sieve opening (μm)	Percentage finer than based on total sample		
	Commercial 1	Commercial 2	Residential
12,700	100.0	100.0	100.0
6,350	56.6	69.0	93.4
3,175	38.2	57.1	82.6
1,999	24.1	40.5	64.3
841	23.5	39.8	60.5
419	15.5	24.9	32.8
249	10.8	14.6	14.8
150	7.6	8.9	5.5
74	4.8	4.4	1.9
Pan	2.2	1.2	0.6

Table 10 Sieve results for the second part of the study

Sieve opening (μm)	Percentage finer than based on total sample							
	Commercial 1					Residential 1		
	1	2	3	4	5	1	2	3
12,700	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
6,350	49.7	42.4	33.9	79.6	65.9	97.0	49.1	29.8
3,175	38.5	32.8	25.5	66.7	55.1	89.5	31.1	19.1
1,999	25.1	24.3	19.1	53.4	44.9	76.0	20.8	11.0
841	24.1	23.3	18.9	51.0	37.3	72.3	19.7	10.6
419	13.3	21.3	14.7	30.3	20.1	43.6	9.4	7.1
249	7.2	17.8	10.4	14.2	15.6	17.8	3.3	3.9
178	3.7	12.4	7.4	5.8	9.7	6.3	1.2	1.8
150	2.2	8.5	5.9	3.2	6.3	2.8	0.5	1.0
74	1.6	6.5	5.0	2.3	4.7	1.6	0.3	0.7
Pan	0.5	2.1	2.6	0.8	1.3	0.3	0.1	0.2

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Table 11 Unit loading rates of collected material (kg/m^3 of runoff)

Size (μm)	Commercial 1	2	3	4	5	Residential 1	2	3
> 12,700	0.92	1.24	2.06	0.68	0.82	0.62	0.17	0.11
12,7000 – 6,350	0.20	0.21	0.26	0.43	0.26	0.22	0.03	0.28
6,350 – 3,175	0.25	0.18	0.20	0.44	0.24	0.13	0.02	0.50
< 3,750	0.46	0.52	0.60	1.79	1.08	0.25	0.03	2.84
Total	**1.83**	**2.15**	**3.12**	**3.34**	**2.40**	**1.22**	**0.25**	**3.73**

captured litter and particles. Table 11 shows these results. The coefficients are shown in units of kg/m^3. Note that the solids larger than 12,700 μm are included. These coefficients were calculated using the catchment area for each site, rainfall observed during the study, and runoff coefficients of 0.39 for residential and 0.6–0.7 for commercial sites. These totals include material swept or blown into the catch basin during non-rainy periods, which in Southern California is the majority of the time. The coefficients in Table 11 will have two systematic errors. The coefficients will be lower than the actual load, since the insert devices are imperfect and bypass at high flow. The coefficients are higher than the actual load carried by stormwater, due to the flux of material in dry weather. The coefficients can be used as a first-order approximation of the litter and debris to be expected from commercial and residential sites in urban areas in climates similar to Los Angeles.

Conclusions

This manuscript has briefly described the results of laboratory and field tests to determine the opportunities for using catch basin inserts to remove specific pollutants (oil and grease, litter and suspended solids). The inserts have the advantage of using the existing urban infrastructure to remove stormwater pollutants at low cost. The estimated cost of each insert is less than US$ 500. An insert design has been proposed that is easy to install and does not require workers to enter the catch basin. Observations during storms showed that they do not create flooding problems, even when they are clogged. Laboratory testing has showed that free oil and grease (simulated by used automobile crankcase oil) can be removed by a variety of sorbents in simple flow-through contacters. Emulsified oil can generally not be removed. Oil and grease removal in field tests was inconclusive. Laboratory testing showed that particles can be removed down to a size of 100 μm, and field results showed that much smaller particles can also be trapped. Laboratory testing showed that the sorbents can remove dissolved PAHs with efficiencies ranging from 16 to 88%. Additional testing is needed to further demonstrate the utility of these inserts. The removal capacities for oil and grease and suspended solids, which will dictate maintenance frequency and cost, need to be determined. The results presented in this paper are preliminary and should be applied with caution. The authors hope that they will stimulate others to develop catch basin insert technology.

Acknowledgements

This work was supported in part by a US-EPA funded Watershed project, the Santa Monica Bay Restoration Project and a consortium of local cities, led by the City of Santa Monica. Additional funding was provided by two manufacturers. The patents or potential patents that cover these products are not known to the authors. There is no financial or business relationship between the authors, UCLA and any insert or sorbent manufacturer. We are thankful to Lee-Hyung Kim, Ed Zaruba, Ruta Skirrus, and the Woodward-Clyde team for their assistance.

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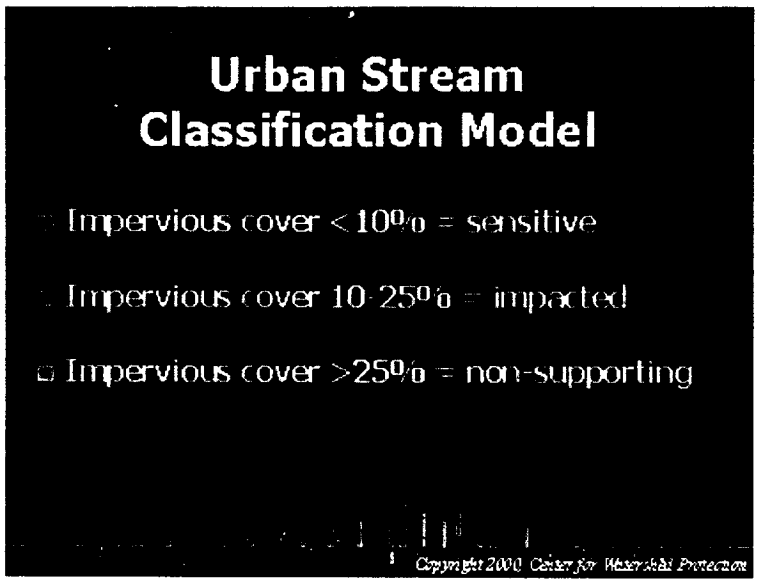

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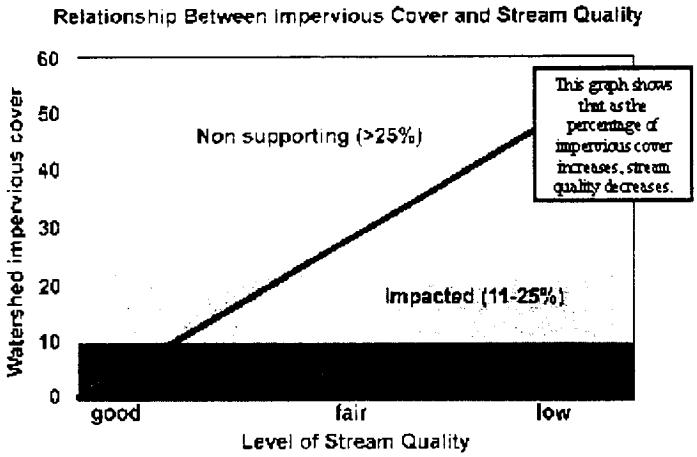
Notes:

A simple model to classify and manage urban streams can be constructed based on impervious cover. The model helps determine the impacts of impervious cover based on three stream categories: sensitive, impacted, and non-supporting. When impervious cover is less than 10%, we still can maintain a high quality stream system that maintains stable banks, has high fish diversity and good water quality. Once impervious cover exceeds 10%, the pristine conditions of the sensitive stream cannot be maintained. Impacted streams are characterized by 10% to 25% imperviousness and begin to show the varying degrees of the influence of land development including unstable channels, declining water quality, and diminished biological communities. Non-supporting streams exceed 25 to 30% impervious cover and are characterized as highly unstable, have poor fish and aquatic insect diversity, and very poor water quality.



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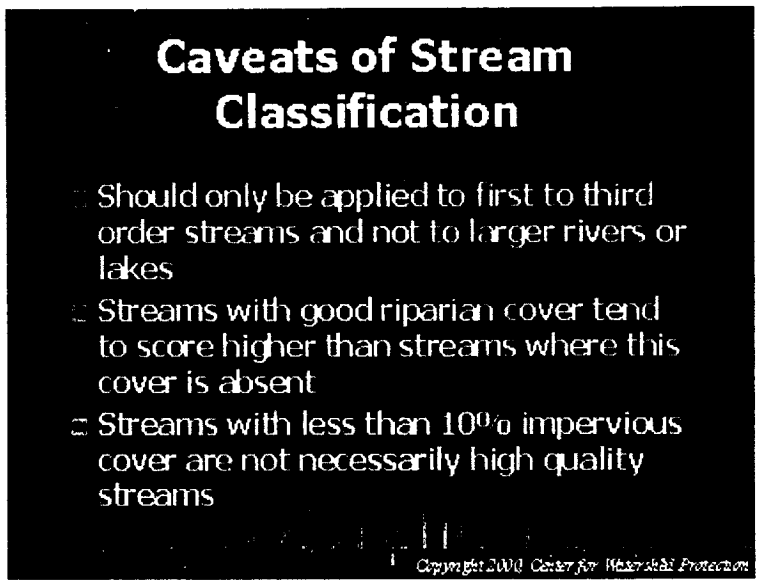
Notes:

This graph shows that as the percentage of impervious cover increases, the amount of sensitive aquatic biota decreases. At less than 10% impervious cover, streams are considered sensitive. Between 10-25%, streams are categorized as impacted. At levels greater than 25% impervious cover, streams are considered non-supporting.



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Notes:

Several caveats need to be considered when using this impervious cover model. To begin with, this model should only be applied to first to third order streams and not to larger rivers or lakes. Secondly, the model indicates that streams with good riparian cover tend to score higher than streams where this is absent. Thirdly, the mere fact that a stream has less than 10% impervious cover does not necessarily mean that it will be a high quality stream, since many of our streams have been historically altered by agriculture, engineering and other impacts such as sedimentation.

Water Quality Sizing (WQ_V)

It is widely recognized that in order to meet various in-stream water quality monitoring standards and classifications, it is necessary to provide some level of stormwater treatment. The water quality volume attempts to capture the majority of the pollutants moving off the land surface. The goal of water quality sizing criteria is to capture and treat the majority pollutants in stormwater runoff, while maintaining a reasonable cost for stormwater treatment practices.

In this section, we present basic options for stormwater quality control, provide default criteria, and discuss tools of analysis to evaluate site pollutant loads.

Basic Options for Water Quality

90 % Rainfall Event:

In this option, the water quality volume is equal to the storage required to capture and treat approximately 90% of the average annual stormwater runoff volume. The specific rainfall event captured is the 90% storm event, or the storm event that is greater than or equal to 90% of all 24-hour storms on an annual basis. This value is determined by investigating local rainfall records to develop a rainfall frequency spectrum. The rainfall frequency spectrum represents the statistical distribution of 24-hour rainfall events. Figure 1 illustrates a rainfall frequency spectrum for Albany, New York. This value varies regionally, based on local rainfall patterns. Table 1 illustrates some typical 90% rainfall events for various cities in the United States.

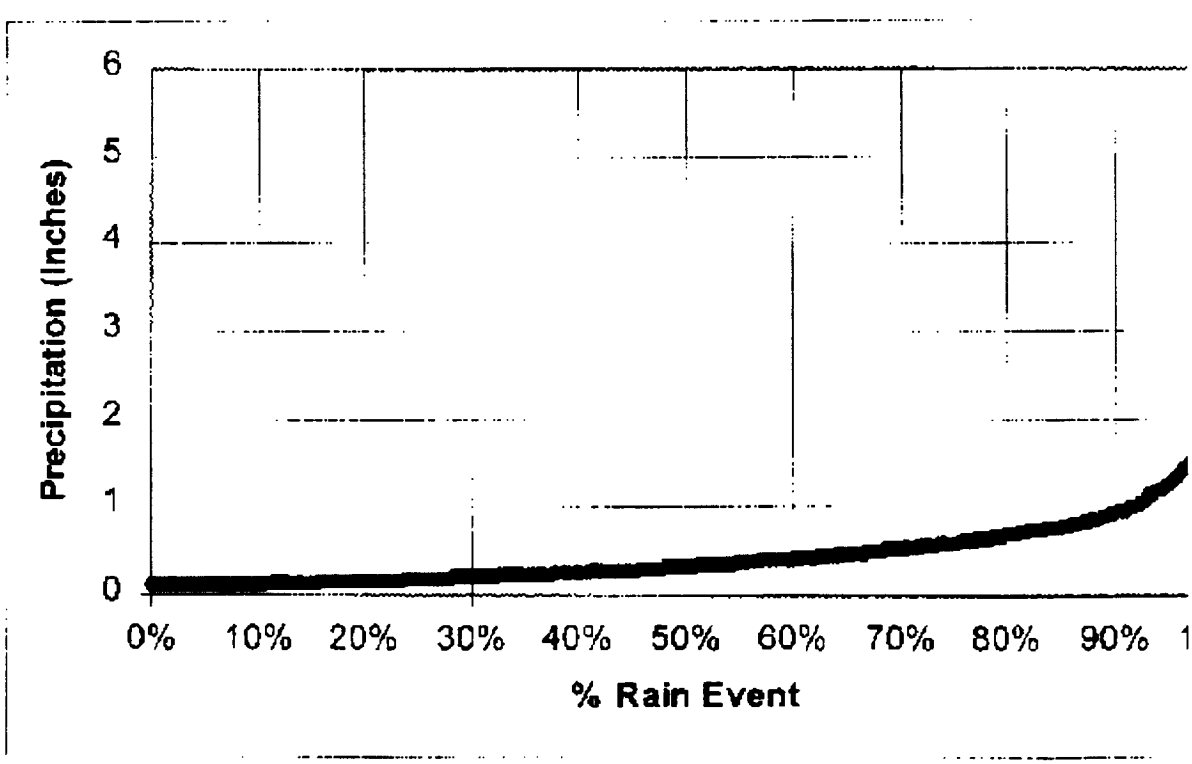

Figure 1. Rainfall Frequency Distribution for Albany, New York

Typically, rainfall events less than 0.1" are eliminated from the rainfall frequency spectrum analysis.

Please note that the 90% rainfall event is not calculated based on the annual volume, but rather as a percentile of individual events. One concern is that this event may not in fact capture 90% of the annual runoff volume. Staff at the Center for Watershed Protection investigated this concern for rainfall stations in New York, Vermont, and Georgia. Assuming that all storms smaller than the water quality storm are captured, and that the water quality volume is captured for larger storm events, the criteria did capture roughly 90% of the annual runoff volume for all stations analyzed. In areas with radically different hydrology, such as regions with frequent hurricanes, or in desert climates, this analysis may not yield the same results.

Table 1. 90% Rainfall Event for Select U.S. Cities

City	Rainfall (Inches)
Columbus, OH	1.0
Albany, NY	0.9
New York, NY	1.2
Frederick, MD	1.1
Washington, D.C.	1.2
Boise, ID	0.5
Phoenix, AZ	0.8
Denver, CO	0.7
Austin, TX	1.4
Savannah, GA	1.5
Montpelier, VT	0.9
Los Angeles, CA	1.3

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When applying the 90% rule to large geographic areas, several stations should be analyzed, because rainfall patterns may vary within the jurisdiction. This analysis may result in different sizing criteria between geographic regions. A stormwater design manual should include a map depicting the required water quality volume required in each geographic region. Figure 2 illustrates a 90% rule map for Maryland.

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One-Inch Sizing:

This option is very similar to the 90% sizing option, except that it applies one rainfall volume arbitrarily. As Table 1 indicates, the actual 90% event may vary significantly between cities.

Half-Inch Rule:

This option is based on the "First Flush" concept, which states that the majority of the pollutants carried in urban runoff are carried in the first half-inch of runoff. This volume is calculated by multiplying 0.5 inches by the total site area. This criterion results in a sizing rule that is perhaps more than is needed for relatively pervious sites, but inadequate for highly impervious ones. One study in Austin, TX (Chang et al., 1990) found that half-inch rule sizing resulted in 100% capture of the total solids load for sites at 10% impervious cover, but only 75% for 50% impervious sites, and 43% for 90% impervious ones.

On-Site Load Calculation:

In this option, the goal is to reduce post development loads by a certain amount. Options used in several areas include reducing pollutant loads for a specific parameter to predevelopment levels, reducing pollutant loads to a certain baseline condition (e.g., the Phosphorus load associated with 15% imperviousness). Examples of these requirements include the Maryland Critical Areas, the New York City Reservoir, and the Chesapeake Bay Resource Protection Areas (VA).

> *Other sizing options for water quality are used throughout the United States. This summary represents only the most common criteria.*

Default Criteria: 90% Event

The WQ_V shall be equal to:

(P1) (Rv)(A)/12

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Where:

P1 = 90% Rainfall Event (Inches)
RV = Runoff Coefficient
A = Site Area (acres)

RV = 0.05+0.009I

Where:

I = Site Impervious Cover (%)

This equation uses the runoff coefficient (RV), described in the . Other simple regressions or methods could be used as a substitute to calculate the runoff volume. Regressions based on local data are preferred.

Tools of Analysis

For quantity-based water quality sizing criteria, several simple regressions can be used to calculate the runoff volume. A few methodologies for methods that calculate on-site loads include:

- The Simple Method is a simple way of calculating runoff and pollutant loads based on impervious cover, rainfall and event mean concentration (EMC) data for different water quality parameters. This model has been expanded to incorporate subsurface flows as well in the Simplified Urban Nutrient Output Model (SUNOM). Click here for a more detailed description of the

- SWIMM is a model developed by the EPA for analyzing stormwater quantity and quality associated with runoff from urban areas. Both single-event and continuous simulation can be performed on catchments having storm sewers, or combined sewers and natural drainage, for prediction of flows, stages and pollutant concentrations.

- SLAMM This model is based on small storm hydrology and pollutant runoff from urban land uses. Pollutant sources are identified and both structural and nonstructural stormwater practices can be accounted for in the model. http://wi.water.usgs.gov/slamm/

This list represents only a small fraction of water quality models. For more general information on water quality models, go to .

Assessment of efficient sampling designs for urban stormwater monitoring

Molly K. Leecaster[1], Kenneth C. Schiff, and Liesl L. Tiefenthaler

ABSTRACT

Monitoring programs for urban runoff, which are highly variable and do not fit a point source model, have not been assessed for effectiveness or efficiency in estimating mass emissions. In order to determine appropriate designs for stormwater, total suspended solids (TSS) and flow information from the Santa Ana River was collected nearly every 15 min for every storm of the 1998 water year. All samples were used to calculate the "true load" and then three within-storm sampling designs (flow-interval, time-interval, and simple random) and five among-storm sampling designs (stratified by size, stratified by season, simple random, simple random of medium and large storms, and the first *m* storms of the season) were simulated. Using these designs, we evaluated three estimators for storm mass emissions (mean, volume weighted, and ratio) and three estimators for annual mass emissions (median, ratio, and regular). Designs and estimators were evaluated with respect to accuracy and precision. The optimal strategy was used to determine the appropriate number of storms to sample annually based upon confidence interval width for estimates of annual mass emissions and concentration. The amount of detectable trend in mass emissions and concentration was determined for sample sizes 3 and 7. Single storms were most efficiently characterized by taking 12 samples following a flow-interval schedule and using a volume-weighted estimator of mass emissions. This design and estimator had the best combination of small bias and standard error. Randomly selecting the medium and large storms within a season achieved the smallest bias for concentration and reasonable bias for estimating mass emissions. This design also attained a small standard error. The ratio estimator most accurately estimated concentration and mass emissions from the simple random sample of medium and large storms, and had low bias over all of the designs. This estimator minimized standard error when coupled with the simple random sample of medium and large storms. Sampling seven storms is the most efficient method for attaining small confidence interval width for annual concentration. Sampling three storms per year allows a 20% trend to be detected in mass emissions or concentration over five years. These results are decreased by 10% by sampling seven storms per year.

INTRODUCTION

Urban runoff is a large source of mass emissions to coastal oceans (Schiff and Tiefenthaler, 2001). Runoff contains pollutants that pose a risk to human health (Haile *et al.* 1999) as well as to indigenous plants and animals (Bay and Schiff 1997). This risk is compounded in southern California where most watersheds are highly developed and precipitation is infrequent, which may result in an increase in the number of sources and pollutant accumulation over longer periods of time prior to highly variable seasonal flows.

Routine monitoring of urban runoff discharges is in its early stages of development, and little consistency or comparability has been achieved among monitoring programs (Schiff 1997). This problem is further compounded by the absence of testing programs to evaluate urban runoff sampling strategies for effectiveness and efficiency; therefore, an optimal program has not been identified. Existing monitoring programs for point sources are inappropriate since stormwater flows and concentrations vary by orders of magnitude in a matter of hours (Cross *et al.* 1992).

The objective of this study is to assess various urban stormwater sampling designs. Data from a comprehensively measured system were subsampled to simulate various strategies, and estimators of mass emissions and

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concentration of total suspended solids (TSS) were compared.

METHODS

Sampling strategies that characterize runoff from the Santa Ana River were evaluated using Monte Carlo simulations taken from a year of continuous stormwater sampling. Three strategies were evaluated for their effectiveness in sampling within storms and five strategies were evaluated for their ability to select storms to sample (among-storm sampling). Three estimates of single-storm mass emissions and concentration and three estimates of annual mass emissions and concentration were considered. Optimal monitoring strategies and estimators were chosen to maximize the accuracy and precision of mass emissions and concentration of TSS. The optimal strategy was then used to determine the appropriate number of storms to sample per year based upon confidence interval width for annual mass emissions and concentration. The amount of detectable trend was also determined for sampling three and seven storms.

Stormwater Sampling and Analysis

Stormwater discharges were sampled for an entire water year (October 1, 1997, to September 30, 1998) on the Santa Ana River at W. 5th Street in Santa Ana, California, the last gauging station before discharge to the ocean. Automated stormwater samplers were installed that logged flow continuously and water quality samples were collected when flow rose above baseline conditions (0-0.7 m^3/s). Samples were collected at 15-min intervals; sampling intervals occasionally were extended to 30 min or an hour during tailing storm flows on extremely large storms when flow and runoff concentrations were changing slowly.

Over 1,700 stormwater samples were collected and analyzed for TSS, representing 90% of the total storm volume discharged during the 1998 water year. The remaining 10% of unsampled volume was a result of stormwater flow lower than the pump intake (7%) and equipment malfunction or breakage (3%). All stormwater samples were stored under refrigeration and analyzed for TSS because they are widely viewed as an indicator of stormwater quality and are correlated with other stormwater quality constituents (Sansalone and Buchberger 1997, Thomson *et al.* 1997). The TSS were analyzed by filtering a 10 to 100 mL aliquot of stormwater through a tarred 1.2 µm (micron) Whatman GF/C filter. The filters plus solids were dried at 60° C for 24 h, cooled, and weighed.

Sampling Designs and Estimators

Three designs for sampling within storms and five designs for sampling among storms were simulated and three estimators of mass emissions and concentration were used for each design. These separate, but dependent, issues were addressed in two steps. First, designs and estimators for use within storms were assessed in unison. Second, sample designs and estimators for annual mass emissions were addressed in unison using the optimal within-storm design and estimator.

The three within-storm sample designs were flow interval, time-interval, and simple random sampling. Flow-interval samples were taken at regular volume intervals (in practice, volume would be predicted beforehand based upon standard hydrologic principles or based upon historical data). This method provided samples that were evenly distributed with respect to volume during the storm; as flows and the volume discharged increased, so did the sample pacing. We considered designs of this type using sample sizes of 4, 8, and 12. Time-interval samples, representing the second design, were taken every 15 min for the first hour of the storm and one per hour thereafter, up to 96 h. This design had a random sample size, determined by the length of the storm. Sampling over time ensured that the samples were taken over the whole time range of the storm. The third design was a simple random sample of sizes 3, 4, 8, 12, and 42. Sample sizes of 4, 8, and 12 flow-paced samples and 42 time-interval sample sizes were chosen to compare designs based upon currently used compositing strategies and mean storm duration (Schiff 1997).

Three estimators of within-storm TSS mass emissions were compared (Table 1). The first estimator was the mean TSS from a storm multiplied by the total storm volume. The second estimator, volume weighted, was the product of the TSS and flow for each sample divided by the sum of sample flows and then multiplied by the total storm volume. This estimator adjusted TSS in a sample by the flow of the river during the sample. The third estimator was a ratio estimator (Cochran 1977). The product of TSS from samples and the ratio of total storm over sample volume was multiplied by total storm volume. This estimator assumed a positive relationship between TSS and volume.

Five designs were considered for sampling among storms: (1) stratified by size of storm (small, medium, and large); (2) stratified by season (early, mid-season, and late); (3) simple random sample from all storms; (4) simple random sample from only medium and large storms; and (5) sampling the first storms of the season up to a specified sample size (first m). Size strata were

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TABLE 1. Within-storm estimators of TSS mass emissions.

Mean $\hat{y}_{j,1} = \left(\sum_{i=1}^{n} \frac{TSS_i}{n}\right)\left(\sum_{i=1}^{N_j} V_i\right)$
Volume-weighted $\hat{y}_{j,2} = \left(\frac{\sum_{i=1}^{n} TSS_i \times V_i}{\sum_{i=1}^{n} V_i}\right)\left(\sum_{i=1}^{N_j} V_i\right)$
Ratio $\hat{y}_{j,3} = \left(\frac{\sum_{i=1}^{n} TSS_i}{\sum_{i=1}^{n} V_i}\right)\left(\sum_{i=1}^{N_j} V_i\right)^2$
$\hat{y}_j$ = Estimate of mass loading for storm j n = Sample size TSS_i = Total suspended solids in sample i N_j = Number of possible samples in storm j V_i = River volume between sample $i-1$ and sample i

created by grouping storms into small (vol < 1 x 10^9 l), medium (1 x 10^9 l < vol < 20 x 10^9 l), and large (vol > 20 x 10^9 l) categories. Seasonal stratification was accomplished by specifying early (September-January), mid (January-February), and late (February-May) season storms. Since stratified sampling requires allocation of the sample sizes among strata, all possible allocations were made for each sample size from 3 to 17 (the number of storms in the 1998 water year). Simple random sampling was achieved by Monte Carlo sampling of 1,000 from all possible samples for each sample size from 3 to 17. Simple random sampling of medium and large storms was accomplished using the same methodology for each sample of sizes 3 to 13, the number of medium and large storms.

Three estimators of annual mass emissions and concentration were considered: median, ratio, and regular (Table 2). The median estimator was the product of the median storm TSS concentration and the total volume of storms for the season. The ratio estimator was the product of the sum of TSS over all sampled storms and the ratio of total season and sample volume times the total season volume. This estimator assumed that TSS per storm was positively related to volume per storm. The regular estimator is the product of the total of TSS for the sampled storms and the ratio of the number of storms in the season to the number of storms sampled. To estimate annual TSS concentration, $\overline{Y}$ instead of concentration, we simply divided $\hat{Y}$ by the population multiplier; $\sum_{j=1}^{M} V_j$ for the median and ratio estimators and for the regular estimator.

Comparison of Designs and Estimators

The sampling designs and estimators were compared with respect to bias and precision. Bias was calculated as the average percentage of difference between the expected estimate and the actual results. The expected estimate was the average of the Monte Carlo samples or all possible samples, depending upon the design. Three designs had no random component: (1) time-interval within-storm design; (2) flow-interval within-storm design; and (3) sampling the first storms among-storm design. In these cases, the expected value was calculated from the one possible realization. The estimate was averaged over the sample size since little difference was found among them. Precision for all design/estimator combinations for various sample sizes was reported as a standard error, calculated as the square root of the variance of all calculated estimates. For the non-random designs, the standard error was calculated as the standard deviation of the data divided by the square root of the sample size.

TABLE 2. Among-storm estimators of annual TSS mass emissions.

Median $\hat{Y}_{median} = Median(\overline{y}_j) \times \sum_{j=1}^{M} V_j$
Ratio $\hat{Y}_{ratio} = \left(\frac{\sum_{j=1}^{m} \hat{y}_j}{\sum_{j=1}^{m} V_j}\right)\left(\sum_{j=1}^{M} V_j\right)$
Regular $\hat{Y}_{regular} = \frac{M}{m} \sum_{j=1}^{m} \hat{y}_j$
$\hat{Y}$ = Estimate of annual mass emissions $\overline{y}_j$ = Estimate of concentration for storm j $\hat{y}_j$ = Estimate of mass emissions for storm j m = Number of storms sampled M = Number of storms V_j = Volume of storm j

The appropriate annual sample size was assessed by comparing the confidence interval width for estimates of annual mass emissions and concentration. A 95% confidence interval was used based upon the optimal strategy.

The amount of detectable trend in annual mass emissions and concentration was calculated using the optimal strategy for sample sizes 3 and 7. A 90% confidence interval and 80% power were used. A linear trend was assumed based upon a regression setup

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(Gerrodette 1987). This approach also assumed that the variability observed during the study is consistent from year to year. The assumption appears warranted, or at least conservative, as this year was an El Niño year that generated both typical and atypical storm patterns that varied tremendously in size and duration relative to historical rainfall patterns in the region.

RESULTS

The flow-interval sampling design with 12 samples provided the least bias of storm mass emissions (Table 3). Smaller sample sizes resulted in larger bias, as did both time-interval and simple random sampling within storms. No design consistently had the smallest standard error, but the standard error decreased as sample size increased.

The volume-weighted estimator was the best overall estimator of storm mass emissions (Table 4). It generally attained smaller bias than either the mean estimator or ratio estimator. The volume-weighted estimator also achieved the smallest standard error. Flow-interval sampling with the volume-weighted estimator estimated storm mass emissions that were too high approximately 65% of the time, whereas the simple random sample with median estimator and time-interval sample with volume-weighted estimator estimated mass emissions that were too low at least 65% of the time (Table 4). As a compromise, the flow-interval design with 12 samples and the volume-weighted estimator were used to characterize storms in the among-storm comparisons to achieve minimum bias with maximum precision.

The simple random sample of all storms or of medium and large storms resulted in the least bias in estimating annual TSS concentration (Table 5), but all designs attained similar bias in estimating annual TSS mass emissions (Table 6). Stratifying the results by season and first storms design resulted in the largest amount of bias in estimating annual TSS concentration. The simple random sample of medium and large storms provided the smallest standard error for concentration (Figure 1) and nearly the smallest standard error for mass emissions (Figure 2). Many design/estimator combinations are not included in these figures since their standard errors ranged from hundreds to thousands of times larger.

No estimator consistently gave the lowest bias in estimating annual TSS concentration (Table 5), but the ratio estimator was least biased, except for the first *m* storms design, for estimating annual TSS mass emissions (Table 6). The ratio estimator with eight or fewer storms sampled provided the smallest standard error for concentration (Figure 1). For mass emissions, the ratio estimator provided the smallest standard error (Figure 2). The regular estimator generally overestimated annual mass emissions by a larger number than did the ratio estimator (Tables 5 and 7).

The confidence interval width for annual concentration narrowed as sample size increased, but did not decrease proportionately for sampling more than seven storms (Figure 3). The confidence interval width for annual mass emissions decreased with increasing sample size, indicating no optimal sample size (Figure 4).

A 20% trend in mass emissions or concentration over five years was achieved by sampling three storms (Figure 5). The percent of detectable trend was reduced by 10 to 30% by increasing the sample sizes from 3 to 7, depending upon the number of years of interest. This relationship was the same for concentration and mass emissions.

DISCUSSION

We were able to assess the most efficient and effective monitoring design based upon a census of urban stormwater runoff for one wet season. The preferred within-storm design was a flow- or volume–paced strategy with the volume-weighted estimator. This estimator utilized the available volume or flow information more precisely than the ratio estimator. The preferred among-storm design was a simple random sample of medium- and large-sized storms using the ratio estimator for annual mass emissions or concentrations. Storm by storm, stronger positive correlation was observed between volume and TSS, thus making the ratio estimator more efficient. This

TABLE 3. Bias and precision of within-storm sampling designs and estimators of TSS mass emissions.

Design	Sample Size	Standard Error (MT)			Bias (%)		
		$\hat{y}_1$	$\hat{y}_2$	$\hat{y}_3$	$\hat{y}_1$	$\hat{y}_2$	$\hat{y}_3$
Flow Interval	4	4,295	2,958	372,019	3	-2	29
Flow Interval	8	3,066	1,450	191,902	3	5	29
Flow Interval	12	2,384	847	98,580	<1	2	24
Time Interval	42	969	135	14,687	-13	-7	7,570
Simple Random Sample	12	1,591	<1	5,207	-3	-100	-91

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TABLE 4. TSS mass emissions estimates (in metric tons) for least biased combinations of within-storm designs (sample size) and estimators.

Storm Number	True TSS Loading	Flow Interval (12) and Volume-Weighted Estimator	Simple Random Sample (12) and Current Estimator	Time-Interval (42) and Volume-Weighted Estimator
3	946	835	1,156	625
5	18,436	19,093	18,137	16,973
6	320	338	296	291
7	5,705	6,179	4,957	4,935
9	21	19	22	22
10	5,278	6,174	5,068	4,667
11	31,846	32,602	35,084	28,433
12	17,967	19,239	13,081	17,967
13	867	947	573	917
14	91,628	93,422	107,015	75,446
15	943	883	900	913
16	81	81	84	74
17	3,090	2,504	2,975	3,097
18	1,556	1,636	1,460	1,531
19	623	593	655	583
21	2,107	2,289	1,718	2,031
22	9,499	10,939	9,399	8,361
Total	190,913	197,773	202,578	166,867

TABLE 5. Percent bias of TSS mass emission estimates for among-storm designs and estimators.

Design	$\hat{Y}_{median}$	$\hat{Y}_{ratio}$	$\hat{Y}_{regular}$
Stratified by Storm Size	-18	3	7
Stratified by Storm Season	-27	0	7
Simple Random Sample of All Storms	-27	4	6
Simple Random Sample of Medium & Large Storms	-18	4	6
Sample First *m* Storms	14	32	-1

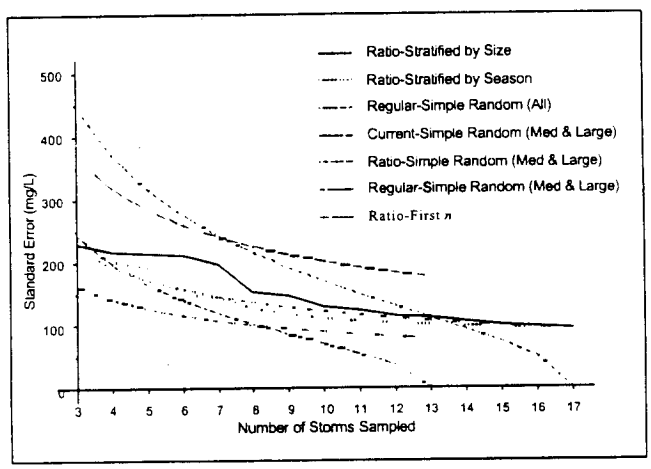
overall combination provided the least bias relative to the actual mass emissions and smallest estimation of standard error.

The optimal strategy provided large improvements on current designs. A variety of designs have been used in southern California (Schiff 1997); one common approach is to use 4 to 12 flow-weighted samples within storms with a mean estimator and a median estimator for annual mass emissions and concentration. The selection of storms for sampling is generally subjective with preference given to large or early storms. This current design has little bias for within-storm estimates but has a standard error almost three times the optimal. The annual estimates of mass emissions and concentration from the median estimator, even with optimal within-storm design, have large negative bias and standard error that are two to three times as large as the optimal design. The correction to achieve more accurate and precise estimates is a simple matter of changing estimators and applying simple modifications to sample and storm selection methodologies.

TABLE 6. Percent bias of TSS concentration estimates for among storm designs and estimators.

Design	$\overline{Y}_{median}$	$\overline{Y}_{ratio}$	$\overline{Y}_{regular}$
Stratified by Storm Size	-6	13	8
Stratified by Storm Season	-17	32	29
Simple Random Sample of All Storms	-27	3	3
Simple Random Sample of Medium & Large Storms	-18	3	-7
Sample First *m* Storms	13	31	45

FIGURE 1. Standard error for estimates of annual TSS concentration for various estimator-sample design combinations.



Although these findings will improve the knowledge of stormwater discharges in an arid region, there are limits to our conclusions. The largest of these limits is our ability to extrapolate our findings to other watersheds. If the variance in water quality changes, or if the relationship between flow and water quality differs, then our conclusions for the Santa Ana River data may not apply. We

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FIGURE 2. Standard error for estimates of annual TSS mass emissions for various estimator-sample design combinations.

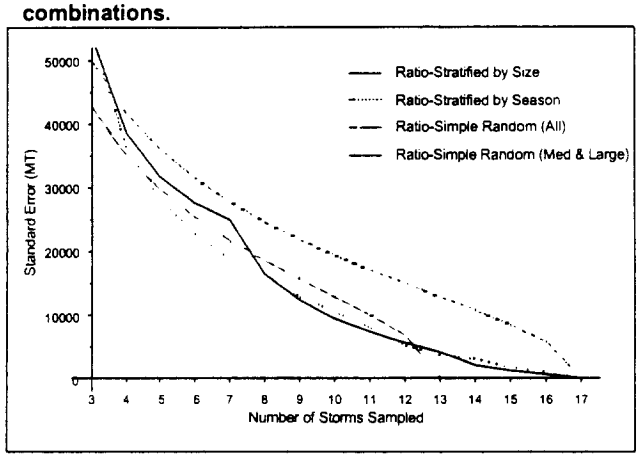


TABLE 7. Annual mass emissions estimates of TSS (in metric tons) for among-storm designs and estimators (number of storms, *m*=7).

Design/ estimator	Annual mass loading (ratio/ estimator)	Annual mass loading (regular/ estimator)
Actual Load= 190,913		
Stratified by Storm Size	197,329	204,432
Stratified by Storm Season	191,418	204,623
Simple Random Sample of All Storms	199,802	203,044
Simple Random Sample of Medium & Large Storms	198,523	202,980
Sample First *m* Storms	253,606	189,796

FIGURE 3. 95% confidence intervals for ratio estimate of annual TSS concentration from simple random sample of medium and large storms (mean = 820 mg/L).

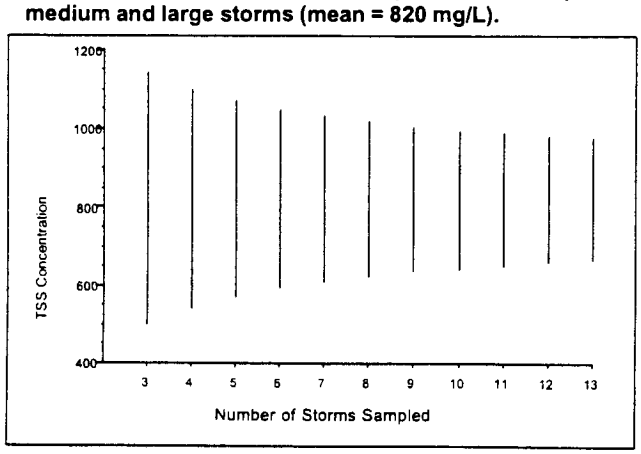


FIGURE 4. 95% confidence intervals for ratio estimate of annual TSS mass emissions from simple random sample of medium and large storms (mass emissions = 6.5 billion metric tons).

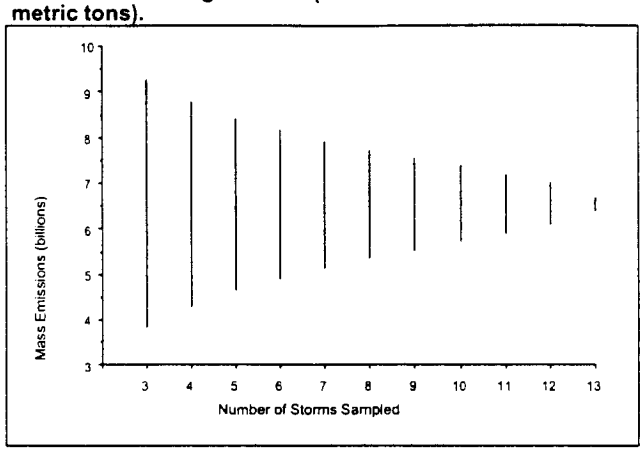
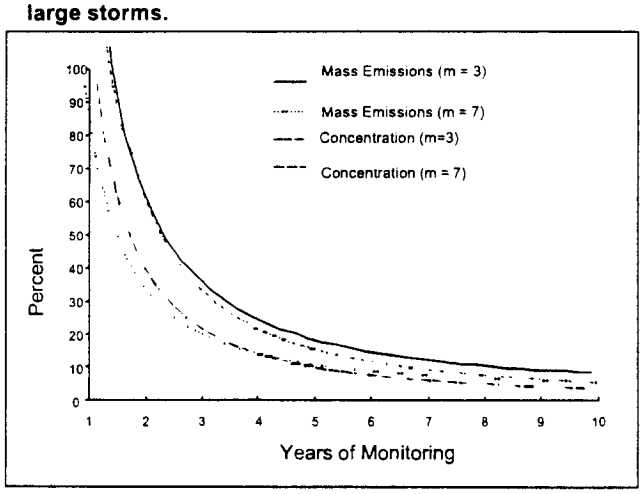


FIGURE 5. Percent detectable trend over years for annual mass emissions and concentration using the ratio estimator with simple random sample of medium and large storms.



suspect that the Santa Ana River is similar to other large, urbanized channels in southern California, but is different from outlets in the rest of the country.

A second limitation is our ability to extrapolate to other years. The 1997/98 season was anomalously wet. Our findings were based upon the wider variability associated with the El Niño effect, so our estimates of standard error were larger also. Because few wet seasons are expected to introduce more variability, we have made conservative estimates of precision and trend detection and therefore suspect that our recommendations would still apply.

Another limitation to this study is the difficulty in extrapolating our conclusions to other constituents. In this study, we utilized TSS since it is an easily measured constituent with relatively small laboratory variability

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suited for exploring different designs. However, we did analyze trace metals, total organic carbon (TOC) and total nitrogen (TN) on a subset of samples and found these constituents to be highly correlated with TSS (Tiefenthaler *et al.*, 2001). Similarly, additional methodologies are available that measure sediment mass emissions and suspended sediments. The methodology utilized in this study was patterned after urban stormwater programs required by National Pollutant Discharge Elimination System Permit reporting and monitoring for municipal separate stormwater sewer systems established by state and federal regulatory agencies (e.g., the U.S. EPA). However, the U.S. Geological Survey (Porterfield 1972) utilizes very different sampling and analytical techniques to estimate fluvial sediment transport. The extent to which these methodologies could alter our assessments of optimal designs is unknown.

The inherent advantage of the volume-paced sampling method was the significant relationship that has been established between flow and TSS in previous studies. In fact, flow accounted for 40% of the variability in TSS concentrations in the Santa Ana River during the 1997/98 wet season (Tiefenthaler *et al., this volume*). Moreover, flow and TSS concentration are highly correlated in all 12 of the largest rivers and creeks in the SCB (Cross *et al.* 1992). Hence, sampling the medium and large storms of the year was a logical endpoint because mass emissions from this river are dominated by the larger episodic events. During 1997/98, more than 95% of the mass emissions from the Santa Ana River occurred during less than 5% of the year; this period was associated with the maximum flows of the wet season.

Trend detection in urban watersheds has been largely overlooked since the necessary data were not available until this present study. Relatively small changes (i.e., 10 to 20%) in both concentration and mass emissions over relatively small time scales (five years or less) can be detected if the monitoring designs are optimized.

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ACKNOWLEDGEMENTS

The authors are extremely grateful for the field and laboratory personnel that assisted with this project: Dario Diehl, David Tsukada, Jeff Brown, Shelly Moore, and Andrew Jirik. A special note of appreciation is given to Harold Stubbs.

R0022860

United States
Environmental Protection
Agency

Office of Research and
Development
Washington, DC 20460

EPA/600/R-92/238
January1993

EPA

Investigation of Inappropriate Pollutant Entries into Storm Drainage Systems

A User's Guide

R0022861

EPA/600/R-92/238
January 1993

INVESTIGATION OF INAPPROPRIATE POLLUTANT ENTRIES INTO STORM DRAINAGE SYSTEMS

A User's Guide

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Contract Number 68-C9-0033 and Cooperative Agreement Number CR-816862

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Printed on Recycled Paper

R0022862

NOTICE

The information in this document has been funded wholly or in part by the United States Environmental Protection Agency under contracts 68-03-3255 and 68-C9-0033 for Foster-Wheeler Enviresponse, Inc. and under cooperative agreement CR-816862 for the Urban Waste Management and Research Center of the University of New Orleans. Although it has been subjected to the Agency's peer and administrative review and has been approved for publication as an EPA document, it does not necessarily reflect the views of the Agency and no official endorsement should be inferred. Also, the mention of trade names or commercial products does not imply endorsement by the United States government.

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FOREWORD

Today's rapidly developing and changing technologies and industrial products and practices frequently carry with them the increased generation of materials that, if improperly dealt with, can threaten both public health and the environment. The U.S. Environmental Protection Agency is charged by Congress with protecting the Nation's land, air, and water resources. Under a mandate of national environmental laws, the Agency strives to formulate and implement actions leading to a compatible balance between human activities and the ability of natural systems to support and nurture life. These laws direct the EPA to perform research to define our environmental problems, measure the impacts, and search for solutions.

The Risk Reduction Engineering Laboratory is responsible for planning, implementing, and managing research, development, and demonstration programs to provide an authoritative, defensive engineering basis in support of the policies, programs, and regulations of the EPA with respect to drinking water, wastewater, pesticides, toxic substances, solid and hazardous wastes, and Superfund-related activities. This publication is one of the products of that research and provides a vital communication link between the researcher and the user community.

The purpose of this User's Guide is to provide guidance to municipalities for investigating non-stormwater entries into storm drainage systems. Contaminated non-stormwater entries into storm drainage systems have been shown to contribute substantial levels of contaminants to the Nation's waterways. These entries may originate from many diverse sources including sanitary wastewaters from leaky or directly connected sanitary sewerage and from poorly operating septic tank systems, washwaters from laundries and vehicle service facilities, and many types of industrial wastewaters that are discharged to floor drains leading to the storm drainage or from direct industrial wastewater connections to the storm drainage system. Conventional pollution control programs may be ineffective if these pollutant sources are not identified and corrected.

This User's Guide will be useful to municipalities in conducting required studies as part of their stormwater discharge permit activities, in addition to other interested users. It will enable users to identify the type and to estimate the magnitude of non-stormwater pollutant entries into storm drainage systems and to design needed pollution control activities. An associated demonstration project (Pitt and Lalor publication pending) describes the development and testing of the procedures presented in this User's Guide.

E. Timothy Oppelt, Director
Risk Reduction Engineering Laboratory

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ABSTRACT

This User's Guide is the result of a series of EPA sponsored research tasks to develop a procedure to investigate non-stormwater entries into storm drainage systems. A number of past projects have found that dry-weather flows discharging from storm drainage systems can contribute significant pollutant loadings to receiving waters. If these loadings are ignored (e.g., by only considering wet-weather stormwater runoff), little improvement in receiving water conditions may occur with many stormwater control programs. These dry-weather flows may originate from many sources, the most important sources may include sanitary wastewater or industrial and commercial pollutant entries, failing septic tank systems, and vehicle maintenance activities. After identification of the outfalls that contain polluted dry-weather flows, additional survey activities are needed to locate and correct the non-stormwater entries into the storm drainage systems.

This User's Guide contains information to allow the design and conduct of local investigations to identify the types and to estimate the magnitudes of these non-stormwater entries.

This report was submitted in partial fulfillment of contracts numbered 68-03-3255 and 68-C9-0033 and cooperative agreement CR-816862 under the sponsorship of the U.S. Environmental Protection Agency. This report covers a period from October 1, 1990 to September 30, 1992, and work was completed as of September 30, 1992. This report was prepared under subcontract to Foster-Wheeler Enviresponse, Inc. of Edison, New Jersey, and the Urban Waste Management and Research Center of the University of New Orleans.

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ACKNOWLEDGMENT

This User's Guide contains information that has been developed and tested in a number of separate research reports investigating inappropriate pollutant entries into storm drainage systems. Many case studies were reviewed during early parts of this research to identify the most appropriate methods of investigation. Information that was obtained from these cities is gratefully acknowledged.

Valuable technical assistance concerning industrial dry-weather discharges was provided by Mark Miller and Tom Meinholtz (Triad Engineering, Inc.) who were supported by Kevin Weiss of the NPDES Branch, Permits Division, Office of Water, of the EPA through the Cadmus Group, Inc. Early report guidance was also provided by Gene Driscoll (Woodward Clyde Consultants), also supported by the Permits Division, Office of Water, of the EPA. Dan Murray, of the Center of Environmental Research Information, Cincinnati, Ohio, EPA, also provided support for the publication of this Guide.

Richard Field, Chief of the Storm and Combined Sewer Pollution Control Program, EPA, was the Project Officer for this project and provided much valued direction during this research. Michael Brown and Marie O'Shea of his staff, along with Ramjee Raghavan at Foster Wheeler Enviresponse, Inc., also provided important project assistance. Darwin Wright of the Office of Research and Development, EPA is gratefully acknowledged for his suggestion to work with the University of New Orleans, Urban Waste Management and Research Center to conduct EPA stormwater research activities. Helpful comments from the report reviewers are also gratefully acknowledged.

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SECTION 1

INTRODUCTION

Current interest in illicit or inappropriate connections to storm drainage systems is an outgrowth of investigations into the larger problem of determining the role urban stormwater runoff plays as a contributor to receiving water quality problems. Urban stormwater runoff is traditionally defined as that portion of precipitation which drains from city surfaces exposed to precipitation and flows via natural or man-made drainage systems into receiving waters. An urban stormwater drainage system also conveys waters and wastes from many other sources. For example, Montoya (1987) found that slightly less than half the water discharged from Sacramento's stormwater drainage system was not directly attributable to precipitation. Sources of some of this water can be identified and accounted for by examining current NPDES (National Pollutant Discharge Elimination System) permit records, for permitted industrial wastewaters that can be discharged to the storm drainage system. However, most of the water comes from other sources, including illicit and/or inappropriate entries to the storm drainage system. These entries can account for a significant amount of the pollutants discharged from storm drainage systems (Pitt and McLean 1986).

The U. S. Environmental Protection Agency's (EPA's) Office of Research and Development's Storm and Combined Sewer Pollution Control Program and the Office of Water's NPDES Program Branch have supported the development of this User's Guide for the investigation of inappropriate entries to storm drainage systems. This User's Guide is designed to provide information and guidance to local agencies by meeting the following objectives of:

1. Identifying and describing the most significant pronounced sources of non-stormwater pollutant entries into storm drainage systems.

2. Describing an investigative procedure that will allow for the determination of whether significant non-stormwater entries are present in a storm drainage system, and then to identify the particular source, as an aid to the ultimate location of the source.

The background study prepared in conjunction with this User's Guide (Pitt and Lalor publication pending) examined three categories of non-stormwater outfall discharges: pathogenic/toxicant, nuisance and aquatic life threatening, and clean water. The most important category is outfall discharges containing pathogenic or toxic pollutants. The most likely sources for this category are sanitary or industrial wastewaters. The outfall analysis procedure described in this User's Guide has a high probability of identifying all of the outfalls in this most critical category. High probabilities of detection of other contaminated outfalls are also likely when using these procedures. After identification of the contaminated outfalls, their associated drainage areas are then subjected to a detailed source identification investigation. The identified pollutant sources are then corrected.

ROLE OF DRY-WEATHER FLOWS IN URBAN STORMWATER RUNOFF ANALYSES

The EPA's Nationwide Urban Runoff Program (NURP) highlighted the significance of pollutants from illicit entries into urban storm drainage (EPA 1983). Such entries may be evidenced by flow from

storm drain outfalls following and during substantial dry periods. Such flow, frequently referred to as "baseflow" or "dry-weather flow", could be the result of direct "illicit connections" as mentioned in the NURP final report (EPA 1983), or could result from indirect connections (e.g., leaky sanitary sewerage contributions through infiltration). Many of these dry-weather flows are continuous and would therefore also occur during rain induced runoff periods. Pollutant contributions from the dry-weather flows in some storm drains have been shown to be high enough to significantly degrade water quality because of their substantial contributions to the annual mass pollutant loadings to receiving waters.

Dry-weather flows and wet-weather flows have been monitored during several urban runoff studies. These studies have found that discharges observed at outfalls during dry weather were significantly different from wet-weather discharges. Data collected during the 1984 Toronto Area Watershed Management Strategy Study (TAWMSS) monitored and characterized both stormwater and baseflows (Pitt and McLean 1986). This project involved intensive monitoring in two test areas (one a mixed residential and commercial area, and the other an industrial area) during both warm and cold weather and during both wet and dry weather. The annual mass discharges of many pollutants were found to be dominated by dry-weather processes.

During the mid-1980s, several individual municipalities and urban counties initiated studies to identify and correct illicit connections to their storm drain systems. This action was usually taken in response to receiving water quality problems or information noted during individual NURP projects. Data from these studies indicate the magnitude of the cross-connection problem in many urban areas. From 1984 to 1986, Washtenaw County, Michigan dye-tested 160 businesses in an effort to locate direct illicit connections to the County stormwater drainage. Of the businesses tested, 61 (38 percent) were found to have improper storm drain connections (Schmidt and Spencer 1986). In 1987, the Huron River Pollution Abatement Program dye-tested 1067 commercial, industrial, and tax exempt businesses and buildings. A total of 154 (14 percent) were found to have improper connections to storm drainage (Washtenaw Co. 1988). Commercial car washes and other automobile related businesses were responsible for the majority of the illicit connections in both studies. Discharges from commercial laundries were also noted. An investigation of outfalls from the separate storm drain system in Toronto, Canada revealed 59 percent with dry-weather flows. Of these, 84 (14 percent of the total outfalls) were identified as grossly polluted based on the results of a battery of chemical tests (GLA 1983). In 1987, an inspection of the 90 urban stormwater outfalls draining into Inner Grays Harbor in Washington revealed 29 (32 percent) flowing during dry weather (Pelletier and Determan 1988). A total of 19 outfalls (21 percent) were described as suspect based on visual observation and/or anomalous pollutant levels as compared to those expected in typical urban stormwater runoff characterized by the EPA 1983 NURP report.

CURRENT LEGISLATION

With additional data now available, the Clean Water Act of 1987 contained provisions specifically addressing discharges from storm drainage systems. Section 402 (p) (3) (B) provides that permits for such discharges:

i. May be issued on a system or jurisdiction-wide basis.

ii. Shall include a requirement to effectively prohibit non-stormwater discharges into the storm drains, and

iii. Shall require controls to reduce the discharge of pollutants to the maximum extent practicable, including management practices, control techniques and system design and

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engineering methods, and such other provisions as the Administrator or the State determines appropriate for the control of such pollutants.

In response to these provisions, the EPA issued a final rule to begin implementation of section 402(p) of the Clean Water Act on November 16, 1990 (40 CFR parts 122, 123, and 124 National Pollutant Discharge Elimination System Permit Regulations for Storm Water Discharges, Federal Register, Vol. 55, No. 222). A screening approach which includes chemical testing of outfalls or storm drainage with dry-weather flow (defined by a 72-hour antecedent dry period), was adopted. The parameters to be tested are a combination of several pollutants of concern and "tracers" that may be used to help identify contaminated outfalls and predict the source of illicit discharges.

Section 122.26 (d) (1) (iv) (D) of the rule applies specifically to this User's Guide. The EPA requires an initial screening program to provide a means of detecting high levels of pollutants in storm sewerage. The protocol of this User's Guide seeks to determine whether or not non-stormwater flows are causing problems (e.g. pathogenic, toxic, aquatic life threatening, nuisance), and to provide additional detail with respect to the source. It accomplishes this by outlining an effective screening methodology to identify storm drainage system outfalls contaminated by illicit or inappropriate discharges and to determine specifically how the likely sources can be identified. This protocol is supported by a research report (Pitt and Lalor publication pending) containing the results of a demonstration project using these procedures and much more detailed information.

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SECTION 2

OVERVIEW

POTENTIAL DRY-WEATHER DISCHARGE SOURCES

This User's Guide is directed to the identification and location of non-stormwater entries into storm drainage systems. It is important to note that for any effective investigation of pollution within a stormwater system, all pollutant sources must be included. Prior research has shown, that for many pollutants, stormwater may contribute the smaller portion of the total pollutant mass discharged from a storm drainage system. Significant pollutant sources may include dry-weather entries occurring during both warm and cold months and snowmelt runoff, in addition to conventional stormwater associated with rainfall. Consequently, much less pollution reduction benefit will occur if only stormwater is considered in a control plan for controlling storm drainage discharges. This User's Guide contains a protocol to identify sources of inappropriate entries to storm drainage systems. The investigations presented in this User's Guide may also identify illicit point source outfalls that do not carry stormwater. Obviously, these outfalls also need to be controlled and permitted.

Table 1 summarizes the potential sources of contaminated entries into storm drainage systems, along with their likely flow characteristics. The following subsections summarize these sources.

Residential and Commercial Sources

The most common potential non-stormwater entries, which have been identified by a review of documented case studies for commercial and residential areas are:

- Sanitary wastewater sources:
  - sanitary wastewater (usually untreated) from improper sewerage connections, exfiltration, or leakage
  - effluent from improperly operating, or improperly designed, nearby septic tanks
- Automobile maintenance and operation sources:
  - car wash wastewaters
  - radiator flushing wastewater
  - engine de-greasing wastes
  - improper oil disposal
  - leaky underground storage tanks
- Irrigation sources:
  - lawn runoff from over-watering
  - direct spraying of impervious surfaces
- Relatively clean sources:
  - infiltrating groundwater
  - water routed from pre-existing springs or streams
  - infiltrating potable water from leaking water mains

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TABLE 1. POTENTIAL INAPPROPRIATE ENTRIES INTO STORM DRAINAGE SYSTEMS

Potential Source:	Storm Drain Entry		Flow Characteristics		Contamination Category		
	Direct	Indirect	Conti-nous	Inter-mittent	Patho-genic/ Toxic	Nuis-ance	Clear
Residential Areas:							
Sanitary Wastewater	X	x	X	x	X	x	
Septic tank effluent		X	X	x	X	x	
Household chemicals	x	X		X	X		
Laundry wastewater	X			X		X	
Excess landscaping watering		X		X	x	x	X
Leaking potable water pipes		X	X				X
Commercial Areas:							
Gasoline filling station	X	x		X	X		
Vehicle maintenance/repair	X	x		X	X		
Laundry wastewater	X		X	x	x	X	
Construction site de-watering		X	X	x		X	
Sanitary wastewater	X	x	X		X		
Industrial Areas:							
Leaking tanks and pipes	x	X	X	x	X		
Miscellaneous process waters[1]	X	x	X	x	X	x	x

Note: X: most likely condition
x: may occur
blank: not very likely

[1] see Table 2 for industrial examples

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- Other sources:
  - laundry wastewaters
  - non-contact cooling water
  - metal plating baths
  - dewatering of construction sites
  - washing of concrete ready-mix trucks
  - sump pump discharges
  - improper disposal of household toxic substances
  - spills from roadway and other accidents
  - chemical, hazardous materials, garbage, sanitary sludge landfills and disposal sites

From the above list, sanitary wastewater is the most significant source of bacteria and oxygen demanding substances, while automobile maintenance and plating baths are the most significant sources of toxicants. Waste discharges associated with the improper disposal of oil and household toxicants tend to be intermittent and low volume. These wastes may therefore not reach the stormwater outfalls unless carried by higher flows from another source, or by stormwater during rains.

Industrial Sources

There are several types of industrial dry-weather entries to storm drainage systems. Common examples include the discharge of cooling water, rinse water, other process wastewater, and sanitary wastewater. Industrial pollutant sources tend to be related to the raw materials used, final product, and the waste or byproducts created. Guidance on typical discharge characteristics associated with common industries is given in Sections 4, 5, and 6.

There is also a high potential for unauthorized connections within older industries. One reason for this is that at the time of an industry's development, sanitary sewers may not have been in existence, since early storm drains preceded the development of many sanitary sewer systems. Also a lack of accurate maps of sanitary and storm drain lines may lead to confusion as to their proper identification. In addition, when the activities within an industry change or expand, there is a possibility for illicit or inadvertent connections, e.g., floor drains and other storm drain connections receiving industrial discharges which should be treated before disposal. Finally, industries processing large volumes of water may find sanitary sewer flow-carrying capacity inadequate or sanitary sewers located too far away, leading to improper removal of excess water through the storm drain system.

Continuous processes, e.g., industrial manufacturing, are important potential sources because any waste streams produced are likely to be constantly flowing. Detection of dry-weather discharges from these sources is therefore made easier, because the continuous and probably undiluted nature of these discharges is more discernable, e.g., odors produced will be stronger and colors more intense along with their tracer constituents being more concentrated and more readily detected by sampling.

Intermittent Sources

The presence of regular, but intermittent, flows will usually be a good indication of contaminated entries to the storm drains, and can usually be distinguished from groundwater infiltration flows. However, as drainage areas increase in size, many intermittent flows will combine to create a continuous composite flow. Examples of possible situations or activities that can produce intermittent dry-weather flows are:

- Wash-up operations at the end of a work shift, or job activity.
- Wash-down following irregular accidents and spills.
- Disposal of process batches or rinse water baths.

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- Over-irrigation of lawns.
- Vehicle maintenance, e.g., washing, radiator flushing, and engine de-greasing.

Industries that operate on a seasonal basis, e.g., fruit canning and tourism can be a source of longer duration intermittent discharges.

Direct Connections to Storm Drains

Direct connections are defined in this Guide as physical connections of sanitary, commercial, or industrial piping (or channels) carrying untreated or partially treated wastewaters to a separate storm drainage system. These connections are usually unauthorized. They may be intentional or may be accidental due to mistaken identification of sanitary sewerlines. They represent the most common source of entries to storm drains by industry.

Direct connections can result in continual or intermittent dry-weather entries of contaminants into the storm drain. Some common situations are:

- Sanitary sewerlines that tie into a storm drain.
- Foundation drains or residential sump-pump discharges that are frequently connected to storm drains. While this practice may be quite appropriate in many cases, it can be a source of contamination when the local groundwater is contaminated, as for example by septic tank failures.
- Commercial laundries and car wash establishments that may route process wastewaters to storm drains rather than sanitary sewers.

Infiltration to Storm Drains

Infiltration into storm drains most commonly occurs through leaking pipe joints and poor connections to catch basins and manhole chimneys but can also be due to other causes, such as damaged pipes and subsidence.

Storm drains, as well as natural drainage channels, can therefore intercept and convey subsurface groundwater and percolating waters. In many cases, these waters will be uncontaminated and have variable flows due to fluctuations in the level of the water table and percolation from rainfall events.

Underground potable water main breaks are another potential clean water source to storm drains. While such occurrences are not a direct pollution source, they should obviously be corrected.

Groundwater may be contaminated, either in localized areas or on a relatively widespread basis. In cases where infiltration into the storm drains occurs, it can be a source of excessive contaminant levels in the storm drains. Potential sources of groundwater contamination include, but are not limited to:

- Failing or nearby septic tank systems.
- Exfiltration from sanitary sewers in poor repair.
- Leaking underground (and above-ground) storage tanks (LUST) and pipes.
- Landfill seepage.
- Hazardous waste disposal sites.
- Naturally occurring toxicants and pollutants due to surrounding geological or natural environment.

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Leaks from underground and above-ground storage tanks and pipes are a common source of soil and groundwater pollution and may lead to continuously contaminated dry-weather entries. These situations are usually found in commercial operations such as gasoline service stations, or industries involving the piped transfer of process liquids over long distances and the storage of large quantities of fuel, e.g., petroleum refineries.

INVESTIGATION METHODOLOGY

Applying the methodology presented in this User's Guide will determine if a storm drain outfall (and drainage system) is affected by pronounced non-stormwater entries. In many cases, the information to be collected by using this methodology will also result in a description of the most likely sources of these discharges.

Several aspects of this methodology were derived from the experience of many municipalities that have previously investigated inappropriate entries into storm drainage systems.

The methodology establishes priorities to identify the areas with the highest potential for causing problems. The investigative procedures then separate the storm drain outfalls into three general categories (with a known level of confidence) to identify which outfalls (and drainage areas) need further analyses and investigations. These categories are outfalls affected by non-stormwater entries from: (1) pathogenic or toxic pollutant sources, (2) nuisance and aquatic life threatening pollutant sources, and (3) unpolluted water sources.

The pathogenic and toxic pollutant source category should be considered the most severe because it can cause illness upon water contact or consumption and significant water treatment problems for downstream consumers, especially if the pollutants are soluble metal and organic toxicants. These pollutants may originate from sanitary, commercial, and industrial wastewater non-stormwater entries. Other residential area sources (besides sanitary wastewater), e.g., inappropriate household toxicant disposal, automobile engine de-greasing, and excessive use of chemicals (fertilizers and pesticides) may also be considered in this most critical category.

Nuisance and aquatic life threatening pollutant sources can originate from residential areas and aside from raw sanitary wastewaters may include laundry wastewaters, lawn irrigation runoff, automobile washwaters, construction site dewatering, and washing of concrete ready-mix trucks. These pollutants can cause excessive dissolved oxygen depletions, and algal growths, tastes and odors in downstream water supplies, offensive coarse solids and floatables, and noticeably colored, turbid or odorous waters.

Clean water discharged through stormwater outfalls can originate from natural springs feeding urban creeks that have been converted to storm drains, infiltrating groundwater, infiltration from potable waterline leaks, etc.

Figure 1 is an outline of the major topics presented in this User's Guide, and Figure 2 is a simplified flow chart for the detailed methodology. The initial phase of the investigative protocol includes the initial mapping and field surveys. These activities require minimal effort and result in little chance of missing a seriously contaminated outfall. The initial activities are followed by more detailed watershed surveys to locate and correct the sources of the contamination in the identified problem areas. After corrective action has been taken, repeated outfall field surveys are required to ensure that the outfalls remain uncontaminated. Receiving water monitoring should also be conducted to analyze water quality improvements. If expected improvements are not noted, then additional contaminant sources are likely present and additional outfall and watershed surveys are needed.

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MAPPING & PRELIMINARY WATERSHED EVALUATION (SECTION 3)

1) Identify receiving waters.
2) Locate all outfalls and associated drainage areas.
3) Compile data on land uses within drainage areas.

SELECTION OF TRACER PARAMETERS (SECTION 4)

1) Select physical and chemical parameters to measure.
2) Determine suitable analysis techniques and number of samples required.
3) Develop library of potential local source flow characteristics.

INITIAL FIELD SCREENING SAMPLING ACTIVITIES (SECTION 5)

1) Conduct outfall screening survey for intermittent and continuous flows.

DATA ANALYSIS TO IDENTIFY PROBLEM OUTFALLS AND FLOW COMPONENTS (SECTION 6)

1) Simple procedures using checklists for typical major flow components.
2) More detailed analyses utilizing library of data on potential source flows will quantify flow components.

WATERSHED SURVEYS TO CONFIRM AND LOCATE INAPPROPRIATE POLLUTANT ENTRIES TO THE STORM DRAINAGE SYSTEM (SECTION 7)

1) Conduct drainage surveys using tracer parameters in critical watersheds.
2) Use flow mass balances, dye studies, smoke tests, and T.V. surveys in isolated drainage areas.

CORRECTIVE TECHNIQUES (SECTION 8)

1) Educate public/industry and enforce with ordinances, zoning, etc.
2) Disconnect illicit direct connections.
3) Wide spread entries may require regional solutions or designation of storm drainage system as a CSO.

Figure 1. Outline of major topics presented in this User's Guide

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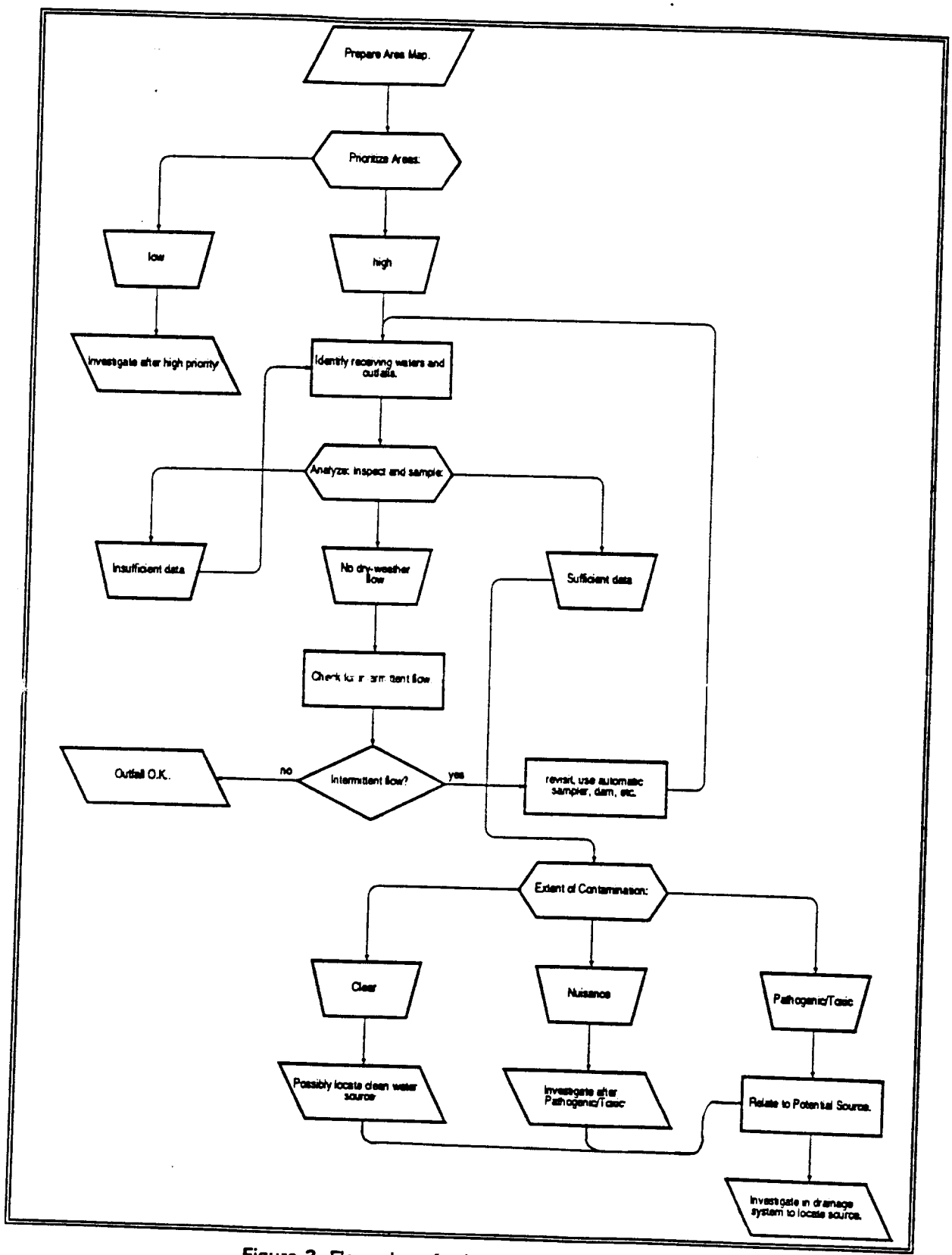

Figure 2. Flow chart for investigation procedures.

RECOMMENDATIONS

This User's Guide should be used as part of a comprehensive stormwater management plan which addresses all sources of stormwater pollution. Correction of pollutant entries identified by use of only this User's Guide is unlikely to achieve a significant improvement in the quality of stormwater discharges or receiving waters.

A municipality will need to plan their investigation of inappropriate entries to a storm drainage system to suit local conditions. This User's Guide describes the issues in sufficient depth and provides examples to enable the design of a local investigation. Greater detail and the results of a comprehensive demonstration of these procedures will be given in a supporting research report by Pitt and Lalor (publication pending).

The full use of all of the applicable procedures described in this User's Guide is likely to be required for successful identification of pollutant sources. Attempting to reduce costs, for example by only examining a certain class of outfalls, or using inappropriate testing procedures, will significantly reduce the utility of the testing program and result in inaccurate data. Also cursory data analyses is likely to result in inaccurate conclusions.

During investigations of non-stormwater entries to storm drainage systems, consideration should be given to any economic and practical advantages of designating the storm drainage system as a combined sewer systems and applying end-of-pipe combined sewer overflow (CSO) control-treatment.

It is also recommended that the methodology (appropriately modified) be applied to other types of sewerage systems, such as combined and separate sanitary sewerage systems, to locate inappropriate entries, e.g., untreated or toxic industrial wastewaters/wastes or infiltration/inflow (I/I) in separate sanitary sewers.

It is recommended that this User's Guide be updated and refined by incorporating experience gained in its application. Incorporation of information from a wide variety of test locations (e.g., lake and large river receiving waters, tidal receiving waters, areas experiencing long dry periods, areas having short summers, areas having unusual groundwater characteristics, etc.) will improve the testing and data analyses protocols described.

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SECTION 3

MAPPING AND PRELIMINARY WATERSHED EVALUATION

PURPOSE

An investigation of non-stormwater entries into a storm drainage system needs to proceed along a systematic path of action, which investigates areas from high to low potential for causing problems, and focuses in from general outfall screening to pin-pointing pollutant sources.

A mapping and evaluation methodology, as detailed in this section, is required to identify the areas to investigate and to provide a basis to prioritize the areas by potential to contribute non-stormwater entries into the storm drainage system.

The data collected in this phase is important as it forms the basis for the rest of the more detailed investigations, described in the subsequent sections of this User's Guide.

MAPPING

To make this exercise as economical and productive as possible, full advantage should be taken of any existing and available information. Data gained from existing sources will need to be supplemented with information obtained by field investigations. The following summarizes the information required, likely data sources, and how to obtain the information.

Receiving Waters and Storm Sewer Outfalls

The receiving waters and stormwater drainage outfalls must be identified and accurately located on appropriate maps. However records of all outfalls are hard to locate, and even for those that can be found, the locations of the outfalls may not be accurate. It is therefore important that the field survey described in Section 5 be used to supplement the data collected during this initial stage. As noted in Section 5, it can take three visits to a drainage area to find all (or almost all) outfalls.

Possible sources of documented information include:

- City records, drainage maps, and storm drain maps.
- Previous surveys, e.g., sanitary sewer infiltration/inflow (I/I) and sewer system evaluation survey (SSES) studies.
- Topographic maps.
- Existing GIS (Geographic Information System) data.
- Pre-development stream locations.
- Drainage department personnel having knowledge of the area.
- Aerial surveys.

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Drainage Area for Each Outfall

The drainage area for each outfall must be determined and marked on the map. This will enable known potential pollutant source locations to be assigned to the correct outfall. Sources for this information are storm drain maps and topographical maps. These should be at least 1" = 200' scale and have no greater than 5 ft contour intervals (depending on the steepness of the area).

Land Uses for Each Outfall Drainage Area

Local planning departments should have detailed zoning maps of the area. These maps should designate residential, commercial, and industrial land uses in each of the outfall drainage areas. In addition, local revenue departments should have lists of business licenses for the entire municipality, but they may not be usefully sorted. The public health department should know where septic tanks are used. Aerial photographs can provide useful information to identify and/or confirm land use areas. Historical land uses, especially landfills and industrial areas, should also be noted.

An effective way to obtain this information is to examine the municipality's zoning maps and to drive to the critical areas to conduct inspections. The land uses of most interest are all industrial, most commercial, and some municipal activities. The activities in the commercial areas of most concern include vehicle related activities (sales, parts, service, or repair), laundry or dry cleaning (including hospitals and hotels), and restaurants. The municipal activities of most concern include but are not limited to: landfills, bus barns, airports, and sanitary wastewater treatment facilities.

Table 2 can be used to identify the local industries in each drainage area most likely to contribute non-stormwater entries into the storm drainage system. The categories considered in this table include loading and unloading of dry bulk or liquid materials, outdoor storage or processing, water usage (cooling and process waters), dust or particulate generating processes, and illicit or inadvertent industrial connections. The likelihood of an industry producing dry-weather or wet-weather discharges in each of these categories was rated on the basis of high, moderate, or low potential and not applicable if there was no relationship evident.

The industrial categories listed in Table 2 were defined according to the 1987 Standard Industrial Classification Manual codes (SIC code). The industries were classified according to six main categories. The category for "Primary Industries" includes facilities involved in the production of food products and other basic goods. The category of "Material Manufacturing" includes those industries producing materials such as lumber, paper, glass, and leather. Similarly, the "Chemical Manufacturing" category includes those industries making products such as plastics, paints, detergents, fertilizers, pesticides, and other related substances. "Transportation and Construction" primarily concerns the discharge of contaminants from building or other types of outdoor development. The "Retail" category includes establishments engaged in the selling of merchandise or offering merchandise related services. Finally, all other industries which did not fit into any of the above classifications were placed into a "General" category. Those industries which are not specifically listed should have characteristics resembling the industries of the major groups with which they are classified by SIC code.

Investigators should take care to include any area where the land use has a potential to contribute pollutant sources to a storm drainage system. As stated above, these land uses may not be covered by Table 2. Some common examples of land use areas to be included are given below:

- Landfill areas can be a source of leachate and polluted runoff.
- Airports have a high potential for fuel spillage. Aircraft deicing agents, and other maintenance operations, produce wastewaters that may be discharged into the storm drainage system.

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TABLE 2. SOURCES OF INDUSTRIAL NON-STORMWATER ENTRIES INTO STORM DRAINAGE SYSTEMS

Industrial Categories Major Classifications SIC Group Numbers	Loading/Unloading Dry Bulk	Loading/Unloading Liquids	Outdoor Storage/ Processing	Water Usage Cooling	Water Usage Process	Particle Generating Process	Illicit/ Inadvertent Connections
Primary Industries							
20 Food & Kindred Products							
201 Meat Products	H	L	H	H	H	L	H
202 Dairy Products Processing Industry	H	H	NA	H	H	NA	H
203 Canned & Preserved Fruits & Vegetables	H	H	H	H	H	M	H
204 Grain Mill Products	H	H	L	H	H	H	H
205 Bakery Products	H	M	NA	NA	H	M	L
206 Sugar & Confectionary Products	H	M	NA	L	M	H	L
207 Fats & Oils	H	H	NA	M	H	NA	M
208 Beverages	H	H	NA	H	H	M	L
21 Tobacco Manufactures	H	M	NA	NA	M	H	M
22 Textile Mill Products	H	L	NA	H	H	M	H
23 Apparel & Other Finished Products Made from Fabrics & Similar Materials	H	L	NA	NA	M	M	L
Material Manufacture							
24 Lumber & Food Products	H	L	H	NA	M	H	L
25 Furniture & Fixtures	H	M	NA	NA	L	M	L
26 Paper & Allied Products	H	H	H	H	H	H	H
27 Printing, Publishing, & Allied Industries	H	M	NA	NA	M	H	L
31 Leather & Leather Products	H	H	L	L	H	H	H
32 Stone, Clay, Glass, & Concrete Products	H	M	H	L	H	H	L
33 Primary Metal Industries	H	M	H	H	H	H	H
34 Fabricated Metal Products	H	H	L	H	H	H	H
37 Transporation Equipment	L	H	L	H	H	L	H

(continued)

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TABLE 2. (continued)

Industrial Categories Major Classifications SIC Group Numbers		Loading/Unloading Dry Bulk	Loading/Unloading Liquids	Outdoor Storage/ Processing	Water Usage Cooling	Water Usage Process	Particle Generating Process	Illicit/ Inadvertent Connections
Chemical Manufacture								
28	Chemicals & Allied Products							
281	Industrial Inorganic Chemicals	H	H	NA	H	H	H	H
282	Plastic Materials & Synthetics	H	H	L	H	M	L	H
283	Drugs	L	L	NA	H	M	L	L
284	Soap, Detergents, & Cleaning Preparations	H	H	NA	H	H	H	H
285	Paints, Varnishes, Lacquers Enamels & Allied Products	H	H	NA	L	H	H	L
286	Industrial Organic Chemicals	H	H	NA	H	H	H	M
287	Agricultural Chemicals	L	L	NA	H	L	L	L
29	Petroleum Refining & Related Industries							
291	Petroleum Refining	L	H	H	H	L	NA	H
295	Paving & Roofing Materials	H	H	H	NA	M	M	L
30	Rubber & Misc. Plastic Products	H	H	NA	H	H	H	M
Transportation & Construction								
15	Building Construction	M	L	H	NA	L	H	L
16	Heavy Construction	M	L	H	NA	L	H	L
Retail								
52	Building Materials, Hardware Garden Supply, & Mobile Home Dealers	H	L	H	NA	L	NA	L
53	General Merchandise Stores	H	M	L	NA	L	NA	L
54	Food Stores	H	H	NA	NA	M	L	L
55	Automotive Dealers & Gasoline Service Stations	H	H	H	NA	M	L	M
56	Apparel & Accessory Stores	H	L	NA	NA	L	NA	L
57	Home Furniture, Furnishings and Equipment Stores	H	L	L	NA	L	NA	L
58	Eating & Drinking Places	H	M	NA	NA	M	NA	M
Other								
	Coal Steam Electric Power	H	L	H	H	L	H	L
	Nuclear Steam Electric Power	NA	L	NA	H	L	NA	NA

NOTE: H: High potential M Medium potential L Low potential NA: Not applicable

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- Government facilities, such as military bases, may store or use polluting materials and have large vehicle maintenance facilities.
- Agricultural impacts are likely to be greater for wet-weather flows, but practices such as irrigation and drainage tiles may also produce dry-weather flows.

Finally, it is necessary to identify and locate existing permitted discharges to streams and storm drainage. The National Pollutant Discharge Elimination System (NPDES) permits, administered by most states or, if not, by the EPA Regional Offices, contain this information for the facilities currently having discharge permits. Only a small fraction of all industries have NPDES permits, as most have no direct wastewater discharges to waters of the United States. Pretreatment programs for municipal sewage treatment plants would also contain additional industrial information.

Other Relevant Information and Features

It is important that investigators be aware of any relevant features or information which may be specific to their drainage area and not included specifically in the above subsections of this User's Guide. Examples of some items that need to be included are discussed in this subsection.

Information on pre-development streams and springs, which may have been routed into the storm drainage system, will aid in the identification of natural uncontaminated or contaminated dry-weather flows.

Information regarding depth to the water table will be helpful. If the water table is well below the storm drain invert at all times, then groundwater infiltration may be less important as a potential source of dry-weather flow. However, the accumulation of percolating shallow groundwater will still occur in storm drainage fill material and be a potential source of some infiltration water. Groundwater conditions for the study area may be available from special studies conducted by the USGS (U.S. Geological Survey), the state water agency, or other sources. Utility construction and repair crews and earth moving companies should know of areas having shallow groundwater. Local I/I and SSES studies also include information concerning shallow groundwater. Well log data collected during drilling of water supply wells, and information from geotechnical investigations, may also be useful.

Areas serviced by sanitary sewerage and areas serviced by septic tanks should be determined in order to identify the areas most likely to have direct connections and infiltration sources, respectively. Either local health, sewerage, utility, environmental, or public works departments should have information on the location of these areas.

Older residential areas with failing infrastructure (especially sanitary sewerage in poor condition), and high density residential areas with septic tanks, should be designated as areas with a high potential for pollutant entries into the storm drainage system.

PRELIMINARY WATERSHED EVALUATION

The above activities should produce maps with complete descriptions of the drainage areas, including outfall locations, NPDES permittees, critical land uses, drainage boundaries for each outfall, city limits, major streets, streams, etc. The investigators need to classify drainage areas by their potential for causing non-stormwater entries. This mapping information, together with the information to be obtained as described in Sections 4 and 5 and analyzed as described in Section 6, will form the basis to rank the drainage areas in order of priority for further detailed drainage area investigations (Sections 7 and 8).

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The investigation of non-stormwater entries will have a cost associated with it, which will increase with the drainage system size and complexity, and with the number of sources being investigated. All pollutant sources, including both wet- and dry-weather pollutant entries, will need to be controlled to have an effective improvement in the quality of the stormwater system discharge. Pitt and McLean (1986) noted that even with the removal of directly connected non-stormwater entries, stormwater originating from industrial and commercial land uses has a high probability of having unacceptable pollutant loads. It would therefore be prudent, at an early stage in the investigation, to review the costs of the investigation and corrective action versus the cost for treatment of the stormwater system discharge. The classification of the storm drainage system as a combined sewer, and subsequent treatment of the flow, may prove to be a more economical and practical alternative. An appropriate time for such a review would be after the mapping and field screening activities to avoid complex, costly, and time consuming drainage system investigations into inappropriate non-stormwater entries, and instead direct resources to pollution control.

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SECTION 4

SELECTION OF TRACER PARAMETERS

INTRODUCTION

The detection and identification of inappropriate entries requires the quantification of specific characteristics of the observed outfall baseflow. The characteristics of most interest should be relatively unique for each potential flow source. This will enable the presence of each flow source to be noted, based on the presence (or absence) of these unique characteristics. The selected characteristics are termed tracers, because they have been selected to enable the identification of the sources of these waters.

One approach presented in this User's Guide is based on the identification and quantification of clean baseflow and contaminated components. If the relative amounts of potential components are known, then the importance of the baseflow can be determined. As an example, if a baseflow is mostly uncontaminated groundwater, but contains 5 percent raw sanitary wastewater, it would be a likely important source of pathogenic bacteria. Typical raw sanitary wastewater parameters (e.g., BOD_5 or suspended solids) would be in low concentrations and the sanitary wastewater source would be difficult to detect. Fecal coliform bacteria measurements would not help much because they originate from many possible sources. Expensive specific pathogen measurements would be needed to detect the problem directly.

The ideal tracer should have the following characteristics:

- Significant difference in concentrations between possible pollutant sources;
- Small variations in concentrations within each likely pollutant source category;
- A conservative behavior (i.e., no significant concentration change due to physical, chemical or biological processes); and,
- Ease of measurement with adequate detection limits, good sensitivity, and repeatability.

In order to identify tracers meeting the above criteria, literature characterizing potential inappropriate entries into storm drainage systems was examined. Several case studies which identified procedures used by individual municipalities or regional agencies were also examined. Though most of the investigations resorted to expensive and time consuming smoke or dye testing to locate individual illicit pollutant entries, a few provided information regarding test parameters or tracers. These screening tests were proven useful in identifying drainage systems with problems before the smoke and dye tests were used. The case studies also revealed the types of illicit pollutant entries most commonly found in storm drainage systems.

This list of potential illicit sources (see Section 2) led to a search for information regarding the chemical and physical characteristics of these specific flows. This search yielded typical characteristics for sanitary wastewater, septic tank effluent, coin-operated laundries and car wash effluents as well as potable water and "natural waters". This information, along with specifics obtained from case studies, provided the basis for selecting parameters for further study. Specific analyses will be needed to identify the characteristics of local potential inappropriate entries and uncontaminated water

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sources, as described in this section.

CANDIDATE PARAMETERS

Many different candidate parameters were evaluated before the suggested list was developed (Pitt and Lalor publication pending). It is recommended that the initial field screening effort (in the absence of known commercial and industrial activities in the watershed) include at least:

- Placement of outfall identification number.
- Outfall discharge flow estimate.
- Floatables, coarse solids, color, turbidity, oil sheen, and odor characteristics of discharge and/or receiving nearfield water.
- Other outfall area characteristics, e.g., stains, debris, damage to concrete, corrosion, unusual plant growth, or absence of plants.
- Water temperature.
- Specific conductivity.
- Fluoride and/or hardness concentrations.
- Ammonia and/or potassium concentrations.
- Surfactant concentration and/or fluorescence.
- Chlorine concentration and pH.

If commercial or industrial activities occur in the drainage area, then it is important to add additional parameters (e.g., a toxicity screening procedure and specific metallic and organic toxicant analyses) to the above list.

Most of the screening effort items listed above can be obtained at the outfall location using field procedures. It is much easier, more cost-effective, and much more accurate to collect samples in the field for later laboratory analyses. Analyzing multiple samples for the same parameter is much more efficient than trying to analyze a single sample for many parameters, especially under adverse field conditions.

The selection of the analysis procedures and equipment will depend on many conditions, most notably the expected concentrations in the uncontaminated baseflows and in the potential non-stormwater discharge flows, along with the needed probabilities of detection at the minimum contamination level. A description of the techniques developed as part of this study to help in the selection of the analytical procedures is given later in this section. Other factors affecting procedure selection include ease of use, analytical interferences, cost of equipment, training requirements, and time requirements to conduct the analyses.

Physical Inspection

Estimates of outfall flow rates, and noting the presence of oil sheens, floatables, coarse solids, color, odors, etc. will probably be the most useful indicators of outfall problems. Physical observations of outfall conditions have been noted in case studies to be very useful in determining the significance of contaminated dry-weather flows. There has been a good correlation between storm drains judged contaminated after physical inspection and those judged contaminated after chemical tests at several case studies (e.g., Inner Grays Harbor, Washington, Beyer, et al. 1979 and Pelletier and Determan 1988; Fort Worth, Texas, Falkenbury 1987 and 1988 and Moore and Hoffpauir 1988; and Toronto, Ontario, GLA 1983).

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Odor--

The odor of a discharge can vary widely and sometimes directly reflects the source of contamination. Industrial dry-weather discharges will often cause the flow to smell like a particular spoiled product, oil, gasoline, specific chemical, or solvent. As an example, for many industries, the decomposition of organic wastes in the discharge will release sulfide compounds into the air above the flow in the sewer, creating an intense smell of rotten eggs. In particular, industries involved in the production of meats, dairy products, and the preservation of vegetables or fruits, are commonly found to discharge organic materials into storm drains. As these organic materials spoil and decay, the sulfide production creates this highly apparent and unpleasant smell. Significant sanitary wastewater contributions to a dry-weather flow will also cause pronounced and distinctive odors.

Color--

Color is another important indicator of inappropriate discharges, especially from industrial sources. Industrial dry-weather discharges can have various colors. Dark colors, such as brown, gray, or black, are most common. For instance, the color contributed by meat processing industries is usually a deep reddish-brown. Paper mill wastes are also brown. In contrast, textile wastes are varied. Other intense colors, such as plating-mill wastes, are often yellow. Washing of work areas in cement and stone working plants can cause cloudy dry-weather discharges. Potential dry-weather sources causing various colored contaminated waters from industrial areas include process waters (slug or continuous discharges), equipment and work area cleaning water discharged to floor drains, and spills during loading operations (and subsequent washing of the material into the storm drains).

Turbidity--

Turbidity of water is often affected by the degree of gross contamination. Dry-weather industrial flows with moderate turbidity can be cloudy, while highly turbid flows can be opaque. High turbidity is often a characteristic of undiluted dry-weather industrial discharges, such as those coming from some continual flow sources, or some intermittent spills. Sanitary wastewater is also often cloudy in nature.

Temperature--

Temperature measurements may be useful in situations where the screening activities are conducted during cold months, or in areas having industrial activity. It may be possible to identify an outfall that is grossly contaminated with sanitary wastewater or cooling water during cold weather and possibly to conduct a rough heat balance. Both sanitary wastewater and cooling water could substantially increase outfall discharge temperatures. Elevated baseflow temperatures (compared to baseflows at other outfalls being screened) could be an indicator of substantial contamination by these warmer source flows.

Floatable Matter--

A contaminated flow may also contain floatables (floating solids or liquids). Evaluation of floatables often leads to the identity of the source of industrial or sanitary wastewater pollution, since these substances are usually direct products or byproducts of the manufacturing process, or distinctive of sanitary wastewater. Floatables of industrial origin may include substances such as animal fats, spoiled food products, oils, plant parts, solvents, sawdust, foams, packing materials, or fuel; whereas floatables in sanitary wastewater include fecal matter, sanitary napkins, and condoms.

Deposits and Stains--

Deposits and stains (residue) refer to any type of coating which remains after a non-stormwater discharge has ceased. They will cover the area surrounding the outfall and are usually of a dark color. Deposits and stains often will contain fragments of floatable substances and, at times, take the form of a crystalline or amorphous powder. These situations are illustrated by the grayish-black deposits that contain fragments of animal flesh and hair which often are produced by leather tanneries, or the white

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crystalline powder which commonly coats sewer outfalls due to nitrogenous fertilizer wastes.

Vegetation--

Vegetation surrounding an outfall may show the effects of intermittent or random non-stormwater discharges. Industrial pollutants will often cause a substantial alteration in the chemical composition and pH of the discharge. This alteration will affect plant growth, even when the source of contamination is intermittent. For example, decaying organic materials coming from various food product wastes could cause an increase in plant life. In contrast, the discharge of chemical dyes and inorganic pigments from textile mills could noticeably stunt plant growth, as these dry-weather discharges are often acidic. In either case, when the industrial pollution constituent in the flow ceases, the vegetation surrounding the outfall will continue to show the effects of the contamination.

In order to accurately judge if the vegetation surrounding an outfall is normal, the observer must take into account the current weather conditions, as well as the time of year in the area. Thus, flourishing or inhibited plant growth, as well as dead and decaying plant life, are all signs of pollution or scouring flows when the condition of the vegetation beyond the outfall contrasts with the plant conditions near the outfall. It is important not to confuse the adverse effects of high storm-induced flows on vegetation with highly toxic dry-weather intermittent flows. Poor plant growth could be associated with scouring flows occurring during storms.

Damage to Sewerage/Outfall Structure--

Sewerage structural damage is another readily visible indication of both continual and intermittent industrial dry-weather discharge contamination. Cracking, deterioration, and spalling of concrete or peeling of surface paint, occurring at an outfall are usually caused by severely contaminated discharges, usually of industrial origin. These contaminants are usually very acidic or basic in nature. For instance, primary metal industries have a strong potential for causing sewerage structural damage because their batch dumps are highly acidic. However confusion is possible due to the effects poor construction, hydraulic scour, and old age may have had on the condition of the outfall structure or sewerage system.

Chemical Parameters

Chemical tests are needed to supplement the above described physical inspection parameters. Chemical tests are needed to quantify the approximate components of a mixture at the outfall. In most cases, dry-weather discharges are made up of many separate source flows (e.g., potable water, groundwaters, sanitary wastewater, and automobile washwaters). Statistical analyses of the chemical test results can be used to estimate the relative magnitudes of the various flow sources (as described in Section 6 of this Guide).

Specific Conductivity--

Specific conductivity can be used as an indicator of dissolved solids. Specific conductivity measurements can be conducted with relative ease in the field, while dissolved solids measurements must be made in a laboratory.

The literature indicates that variation in specific conductivity measurements between water and wastewater sources could be substantial enough to indicate the source of dry-weather flow in the storm drainage system. Specific conductance was judged to be a reliable and quick field indicator of general outfall contamination in Toronto (GLA 1983). Observed levels ranged from 25 to 100,000 μS/cm (microSiemens per cm). Specific conductivity levels less than 1000 μS/cm indicated significant levels of rainwater in the drainage. Specific conductivity can be measured quickly, easily and cheaply. For these reasons, it was selected as a parameter for further study.

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Fluoride--

Fluoride concentration should be a reliable indicator of potable water where fluoride levels in the raw water supply are adjusted to consistent levels and where groundwater has low to non-measurable natural fluoride levels. It is common practice for communities to add fluoride to municipal waters to improve dental health. Concentrations of total fluoride in fluoride treated potable waters are usually in the range of 1.0 to 2.5 mg/L.

Fluoride measurements have often been used to distinguish treated waters from natural waters. During the Allen Creek drainage study (Schmidt and Spencer 1986), the fluoride concentrations of dry-weather flows at outfalls were undetectable after most of the known improper connections to storm drains were eliminated. Very few of these improper connections were of sanitary wastewater to the storm drainage. Apparently, most of the non-stormwater discharges were treated potable water.

Hardness--

Hardness may also be useful in distinguishing between natural and treated waters (like fluoride), as well as between clean treated waters and waters that have been subjected to domestic use.

The hardness of waters varies considerably from place to place, with groundwaters generally being harder than surface waters. Natural sources of hardness are limestones which are dissolved by percolating rainwater made acid by dissolved carbon dioxide. Information regarding the average hardness of potable water as well as local groundwater and surface waters should be readily available wherever a public water supply system exists.

Ammonia/Ammonium--

As part of the nitrogen cycle, ammonia is produced by the decay of organic nitrogen compounds. Ammonia may then be broken down, forming nitrites and nitrates. The presence or absence of ammonia (NH3), or ammonium ion (NH4+), has been commonly used as a chemical indicator for prioritizing sanitary wastewater cross-connection drainage problems. Correlations between elimination of improper sanitary wastewater cross-connections into storm drainage and reduced numbers of storm drainage outfalls with ammonia present were noted in Fort Worth (Falkenbury 1987 and 1988; Moore and Hoffpauir 1988). During studies in Toronto (GLA 1983), more "problem" storm drain outfalls had high ammonia concentrations (>1 mg/L) than any other single parameter, except TKN. During the Huron River (Michigan) study (Washtenaw Co. 1987 and 1988; Murray 1985), ammonia levels were found to be greater at all "problem" storm drain outfalls than at control locations. However, the Allen Creek (Michigan) Drainage study (Schmidt and Spencer 1986) reported that with 92 percent of the improper non-stormwater entries to storm drains eliminated, the ammonia concentrations did not change significantly (all were about 0.44 mg/L). However, very few of these cross-connection eliminations were for sanitary wastewater. Ammonia should be useful in identifying sanitary wastes and distinguishing them from commercial water usage.

Potassium--

Large increases of potassium concentrations have been noted for sanitary wastewater compared to potable water during studies in California (Evans 1968), Virginia (Hypes, et al. 1975), and Brussels, Belgium (Verbanck, et al. 1990). These potassium increases following domestic water usage suggest its potential as a tracer parameter.

Surfactants and Fluorescence--

Surfactants are discharged from household and industrial laundering and other cleaning operations. In the United States, anionic surfactants are commonly used in detergents and account for approximately two thirds of the total surfactants used. Anionic surfactants are commonly measured as Methylene Blue Active Substances (MBAS). In raw sanitary wastewaters, surfactants generally range from 1 to 20 mg/L, while natural waters usually have surfactant concentrations below 0.1 mg/L.

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Large concentrations of surfactants are found in sanitary wastewater, but some researchers (Alhajjar, et al. 1989) have reported that they are not found in septic tank effluent. Surfactants can be totally degraded in the septic tanks. During the Allen Creek drainage study (Schmidt and Spencer 1986; Washtenaw County Drain Commissioner 1984; and Washtenaw County Statutory Drainage Board 1987), surfactants (as MBAS) decreased significantly after most of the improper non-stormwater entries to storm drains were eliminated. Surfactants can be used to identify sanitary or laundry wastewater cross-contamination in storm drainage systems. They may also be of use in distinguishing between infiltrating septic tank effluent and other washwaters from domestic or commercial cleaning operations.

Water fluorescence is also an indicator of detergent residue in waters. Most detergents contain fabric whiteners which cause substantial fluorescence. Fluorescent indicators remain after sanitary wastewater treatment in septic tanks. Fluorescence in contrast to MBAS may be useful in distinguishing between sanitary wastewater contamination and septic tank effluent.

pH--

The pH of most uncontaminated baseflows, as well as sanitary wastewater, is usually quite close to neutral (pH of 7). Therefore, Ph will probably not serve as an indicator of sanitary cross connections. However, pH values may be extreme in certain inappropriate commercial and industrial flows or where groundwaters contain dissolved minerals. If unusual pH values are observed, then the drainage system needs to be carefully evaluated. Very few of the stormwater outfalls tested during dry-weather in Fort Worth (Falkenbury 1987 and 1988; Moore and Hoffpauir 1988) had pH values either below 6 or above 9. None of the Toronto (GLA 1983) "problem" outfalls were reported to have extreme pH values.

Chemicals (acidic and alkaline) released into storm drains by chemically-oriented industries are frequently the cause of pH fluctuations which can range from 3 to 12.

Industries that commonly release low pH (acidic) dry-weather discharges include (but are not limited to) textile mills, pharmaceutical manufacturers, metal finishers/fabricators, as well as companies producing resins, fertilizers and pesticides. Wastes containing sulfuric, hydrochloric, or nitric acids are common industrial sources of low pH discharges.

Many industrial wastes contain high pH (alkaline) chemicals such as cyanide, sodium sulfide, and sodium hydroxide. High concentrations of these contaminants are found in discharges from soap manufacturers, textile mills, metal plating industries, steel mills, and producers of rubber or plastic.

Total Available Chlorine--

Chlorine can be present in water as free available chlorine and as combined available chlorine (usually as chloramines). Both types can exist in the same water and be determined together as the total available chlorine. Chlorine is not stable in water, especially in the presence of organic compounds. Tests of clean potable water during the demonstration project (Pitt and Lalor publication pending) found that total available chlorine only decreased by about 25 percent in 24-hours during an aerated bench-scale test. However, the chlorine demand of contaminated water can be very large, with chlorine concentrations decreasing to very small values after short periods of time. Chlorine therefore cannot be used to quantify flow sources because of its instability, but the presence of chlorine in baseflow waters (very unlikely) could indicate a significant and very close potable water flow source.

Other Chemicals Indicative of Manufacturing Industrial Activities--

Table 3 is a listing of various chemicals that may be associated with a variety of different industrial activities. If the industrial activities in an outfall watershed are known, it may be possible to examine the non-stormwater outfall flow for specific chemicals (e.g., listed in Table 3) to identify which industrial activities may be responsible for the dry-weather flow.

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TABLE 3. SIGNIFICANT CHEMICALS IN INDUSTRIAL WASTEWATERS

Chemical:	Industry:
Acetic acid	Acetate rayon, pickle and beetroot manufacture.
Alkalies	Cotton and straw kiering, cotton manufacture, mercerizing, wool scouring, and laundries.
Ammonia	Gas, coke, and chemical manufacture.
Arsenic	Sheep-dipping, and felt mongering.
Chlorine	Laundries, paper mills, and textile bleaching.
Chromium	Plating, chrome tanning, and aluminum anodizing.
Cadmium	Plating.
Citric acid	Soft drinks and citrus fruit processing.
Copper	Plating, pickling, and rayon manufacture.
Cyanides	Plating, metal cleaning, case-hardening, and gas manufacture.
Fats, oils	Wool scouring, laundries, textiles,and oil refineries.
Fluorides	Gas, coke, and chemical manufacture, fertilizer plants, transistor manufacture, metal refining, ceramic plants, and glass etching.
Formalin	Manufacture of synthetic resins and penicillin.
Hydrocarbons	Petrochemical and rubber factories.
Hydrogen peroxide	Textile bleaching, and rocket motor testing.
Lead	Battery manufacture, lead mining, paint manufacture, and gasoline manufacture.
Mercaptans	Oil refining, and pulp mills.
Mineral acids	Chemical manufacture, mines, Fe and Cu pickling, brewing, textiles, photo-engraving, and battery manufacture.
Nickel	Plating.
Nitro compounds	Explosives and chemical works.
Organic acids	Distilleries and fermentation plants.
Phenols	Gas and coke manufacture, synthetic resin manufacture, textiles, tanneries, tar, chemical, and dye manufacture and sheep-dipping.
Silver	Plating, and photography.
Starch	Food, textile, and wallpaper manufacture.
Sugars	Dairies, foods, sugar refining, and preserves.
Sulfides	Textiles, tanneries, gas manufacture, and rayon manufacture.
Sulfites	Wood process, viscose manufacture, and bleaching.
Tannic acid	Tanning, and sawmills.
Tartaric acid	Dyeing, wine, leather, and chemical manufacture.
Zinc	Galvanizing, plating, viscose manufacture, and rubber process.

Source: Van der Leeden, et al. 1990.

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Toxicity Screening Tests

In addition to the parameters described above, relative toxicity can be an important outfall screening parameter. Short-term toxicity tests, such as the Microtox™ test (from Microbics) are valuable for quickly and cheaply assessing the relative toxicity (to a selected test organism) of different storm drain baseflows. These tests can be used to identify outfalls that contain flows in the most serious (toxic) category and that require immediate investigation. These tests are also very useful in identifying likely sources of toxicants to the drainage system by utilizing a toxicity reduction evaluation (TRE) procedure in the drainage system. If an outfall contains a highly toxic flow, then specific metallic and organic toxicants can be analyzed to support source identification.

TRACER CHARACTERISTICS OF SOURCE FLOWS

Table 4 summarizes the relative concentrations of tracer parameters in source flows. The unique "fingerprints" of each flow category shown can be used to identify the flow components, as shown in Section 6. This table also contains redundancies, (e.g., potassium and ammonia) to help identify sanitary wastewater and septic tank effluent. Fluoride and hardness are similarly used to identify treated potable water and surfactant (MBAS) and fluorescent measurements are used to identify washwaters.

Table 5 is a summary of the tracer parameter concentrations found in Birmingham, Alabama, from April 1991 to September 1992. This table is a summary of the "library" that describes the tracer conditions for each potential source category. The important information shown on this table includes the median and coefficient of variation (COV) values for each tracer parameter for each source category. The COV is the ratio of the standard deviation to the mean. A low COV value indicates a smaller spread of data compared to a data set having a large COV value. It is apparent that some of the abstracted and generalized relationships shown on Table 4 did not exist during the demonstration project. This stresses the need for obtaining local data describing likely source flows.

The fluorescence values shown on Table 5 are direct measurements from the Turner™ (Model 111) fluorometer having general purpose filters and lamps and at the least sensitive setting (number 1 aperture). The toxicity screening test results are expressed as the toxicity response noted after 25 minutes of exposure. The Microtox™ unit measures the light output from phosphorescent algae. The I_{25} value is the percentage light output decrease observed after 25 minutes of exposure to the sample. If an outfall sample has a very high light reduction value, it is typically subjected to additional organic and metallic toxicant tests. Fresh potable water has a relatively high response because of the chlorine levels present. Aged, or dechlorinated, potable water has much smaller toxicity responses.

Appropriate tracers are characterized by having significantly different concentrations in flow source categories requiring identification. In addition, effective tracers also need low COV values within each flow category. Table 4 indicates the expected changes in concentrations per category and Table 5 indicates how these expectations compared with the results of an extensive local sampling effort. The study indicated that the COV values were quite low for each category, with the exception of chlorine, which had much greater COV values. The high chlorine COV values reinforce what was previously indicated (under Total Available Chlorine), that chlorine is not recommended as a quantitative tracer to estimate the flow components. Similar data must be collected in each community where these procedures are to be used. The following subsection discusses how the number of samples needed per category can be estimated.

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TABLE 4. FIELD SURVEY PARAMETERS AND ASSOCIATED NON-STORMWATER FLOW SOURCE CATEGORIES

Parameter	Natural Water	Potable Water	Sanitary Wastewater	Septic Tank Effluent	Indus. Water	Wash-Water	Rinse Water	Irrig. Water
Fluorides	-	+	+	+	+/-	+	+	
Hardness Change	-	+/-	+	+	+/-	+	+	+
Surfactants	-	-	+	-	-	+	-	-
Fluorescence	-	-	+	+	-	+	-	-
Potassium	-	-	+	+	-	-	-	-
Ammonia	-	-	+	+	-	-	-	-
Odor	-	-	+	+	+	+/-	-	-
Color	-	-	-	-	+	-	-	-
Clarity	-	-	+	+	+	+	+/-	-
Floatables	-	-	+	-	+	+/-	+/-	-
Deposits/Stains	-	-	+	-	+	+/-	+/-	-
Vegetation Change	-	-	+	+	+	+/-	-	+
Structural Damage	-	-	-	-	+	-	-	-
Conductivity	-	-	+	+	+	+/-	+	+
Temperature Change	-	-	+/-	-	+	+/-	+/-	-
pH	-	-	-	-	+	-	-	-

NOTE:
- \- implies relatively low concentration
- \+ implies relatively high concentration
- +/- implies variable conditions

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TABLE 5. TRACER CONCENTRATION FOUND IN BIRMINGHAM, ALABAMA WATERS (MEAN, STANDARD DEVIATION AND COEFFICIENT OF VARIATION, COV)

	Spring Water	Treated Potable Water	Laundry Waste-water	Sanitary Waste-water	Septic Tank Effl.	Car Wash-water	Radiator Flush Water
Fluorescence (% scale)	6.8 2.9 0.43	4.6 0.35 0.08	1020 125 0.12	250 50 0.20	430 100 0.23	1200 130 0.11	22,000 950 0.04
Potassium (mg/L)	0.73 0.070 0.10	1.6 0.059 0.04	3.5 0.38 0.11	6.0 1.4 0.23	20 9.5 0.47	43 16 0.37	2800 375 0.13
Ammonia (mg/L)	0.009 0.016 1.7	0.028 0.006 0.23	0.82 0.12 0.14	10 3.3 0.34	90 40 0.44	0.24 0.066 0.28	0.03 0.01 0.3
Fluoride (mg/L)	0.031 0.027 0.87	0.97 0.014 0.02	33 13 0.38	0.77 0.17 0.23	0.99 0.33 0.33	12 2.4 0.20	150 24 0.16
Toxicity (% light decrease after 25 min., I^{25})	<5 n/a n/a	47 20 0.44	99.9 <1 n/a	43 26 0.59	99.9 <1 n/a	99.9 <1 n/a	99.9 <1 n/a
Surfactants (mg/L as MBAS)	<0.5 n/a n/a	<0.5 n/a n/a	27 6.7 0.25	1.5 1.2 0.82	3.1 4.8 1.5	49 5.1 0.11	15 1.6 0.11
Hardness (mg/L)	240 7.8 0.03	49 1.4 0.03	14 8.0 0.57	140 15 0.11	235 150 0.64	160 9.2 0.06	50 1.5 0.03
pH (pH units)	7.0 0.05 0.01	6.9 0.29 0.04	9.1 0.35 0.04	7.1 0.13 0.02	6.8 0.34 0.05	6.7 0.22 0.03	7.0 0.39 0.06
Color (color units)	<1 n/a n/a	<1 n/a n/a	47 12 0.27	38 21 0.55	59 25 0.41	220 78 0.35	3000 44 0.02
Chlorine (mg/L)	0.003 0.005 1.6	0.88 0.60 0.68	0.40 0.10 0.26	0.014 0.020 1.4	0.013 0.013 1.0	0.070 0.080 1.1	0.03 0.016 0.52
Spec. Conduct. (μS/cm)	300 12 0.04	110 1.1 0.01	560 120 0.21	420 55 0.13	430 311 0.72	485 29 0.06	3300 700 0.22
Number of Samples	10	10	10	36	9	10	10

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Determining Number of Observations Needed

It is very important to determine the number of observations needed for each tracer parameter for each source category in order to build a useful data library for analyzing the outfall data. This determination is a function of the tolerable error level in the data means and the standard deviations. The following paragraphs briefly describe a method that can be used to estimate the sampling effort needed to develop a useful library of source characteristic data.

Estimating Errors--

One equation that can be used to calculate the number of analyses needed, based on the allowable error is (Cochran 1963):

$$\text{Number of samples} = 4(\text{standard deviation})^2/(\text{allowable error})^2$$

With a 95 percent level of confidence, this relationship determines the number of samples needed to obtain a value within the range of the sample mean, plus and minus the error. Similarly, this equation can be used to predict the 95 percent confidence interval, based on the measured (or estimated) standard deviation and number of samples obtained:

$$\text{Error} = 2(\text{standard deviation})/(\text{number of samples})^{0.5}$$

where the confidence interval is the mean plus and minus the calculated error value.

Example of Log_{10} Transformation--

These equations assume a normal distribution of the data. However, most water quality data needs to be log_{10} transformed before a normal distribution is obtained. As an example, consider a tracer having a COV of 0.23 and a median value of 0 14. The resulting log_{10} transformed standard deviation would be about 0.12. For ten samples, the resulting 95 percent confidence range of the median observation (0.14 mg/L) is:

$$\text{Error} = 2(0.12)/(10)^{0.5} = 0.076 \text{ in } log_{10} \text{ space}$$

The confidence interval is therefore $log_{10}(0.14)$ +/- 0.076, which is -0.778 to -0.930 in log10 space. This results in a conventional 95 percent confidence range of $10^{-0.930}$ (= 0.12) to $10^{-0.778}$ (= 0.17). The error in the estimate of the median value is therefore between 14 and 21% for ten samples. If the original untransformed data were used, the error associated with 10 samples is 15%, within the range of the estimate after log transformations. These results are close because of the low COV value (0.23). If the COV value is large, the need for log transformations increases. Figure 3 (Pitt 1979) shows the approximate sample size needed to obtain different allowable errors for different COV values (using nontransformed data).

The COV value in the above example (0.23) was close to the median COV value for all of the source categories and tracer parameters shown on Table 5. Therefore, about 10 samples per source flow category should generally result in less than a 25 percent error for the median values obtained.

As shown in a later section, narrow confidence intervals are needed in order to estimate the relative mixes of the non-stormwater sources as measured at the outfall. Therefore, much care needs to be taken in order to estimate the characteristics of the potential non-stormwater flow sources, especially the COV values and medians.

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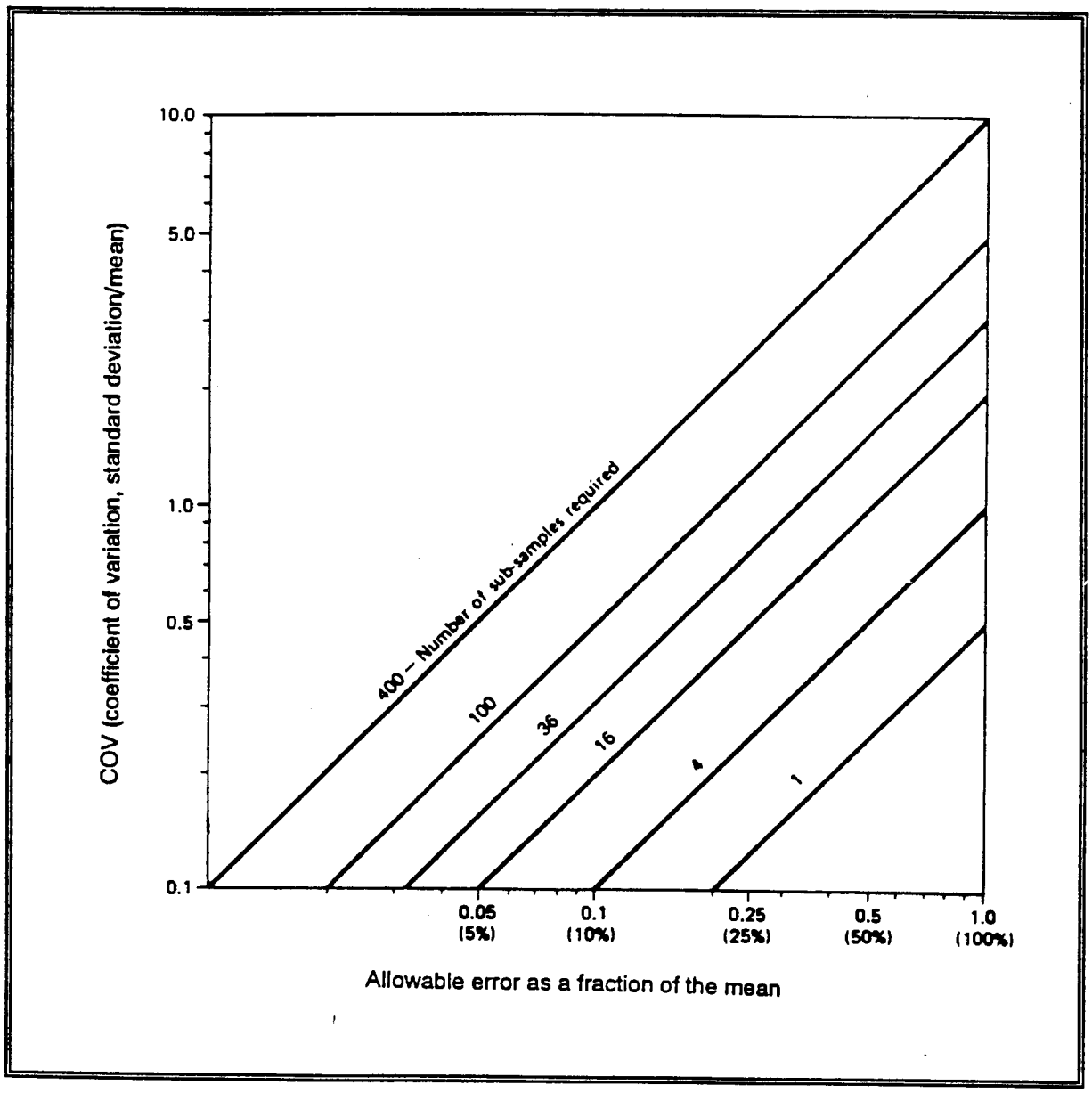

Figure 3. Required number of samples for allowable error and COV

Source: Pitt 1979

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Understanding the mechanisms affecting the non-stormwater sources (e.g., time of day, season, area of town, type and magnitude of land use activities, etc.) and obtaining a relatively large data base library for the source flow tracer concentrations is very important and should be a significant portion of a dry-weather flow source identification project.

SELECTION OF ANALYTICAL METHODS

The selection of the analytical procedure to be used is dependent on a number of factors, including (in order of importance):

- appropriate detection limits
- freedom from interferences
- good analytical precision (repeatability)
- low cost and good durability
- minimal operator training required

The following sub-sections discuss these requirements and present the recommended analytical procedures. Tracer characteristics in potential local source flows affect most of these requirements. Therefore, the suggested analytical procedures may not be the most cost-effective for all areas.

Detection Limit Requirements

In order to identify potential non-stormwater sources, it is necessary to have a basic knowledge about each potential source flow. As shown earlier, a significant sampling and analysis effort is needed to develop a library of source flow tracer concentrations. The COVs and means of the tracer concentrations are needed to estimate the detection limits required by the analytical procedures.

There are a number of different types of detection limits defined for laboratory use. Most instrument manufactures present a minimum readable value as the instrument detection limit (IDL) in their specifications for simple test kits. The usual definition of IDL, however, is a concentration that produces a signal to noise ratio of five. The method detection limit (MDL) is a more conservative value and is established for the complete preparation and analysis procedure. The practical quantification limit (PQL) is higher yet and is defined as a routinely achievable detection limit with a relatively good certainty that any reported value is reliable. Standard Methods (APHA, et al. 1989) estimates that the relationship between these detection limits is approximately: IDL:MDL:PQL = 1:4:20. Therefore, the detection limit shown in much of the manufacturer's literature is much less than what would be used by most analytical laboratories.

Because of the screening nature of the outfall field surveys, the instrument detection capabilities are appropriate for the methodology described in this Users' Guide. The larger uncontrollable errors associated with obtaining representative outfall samples and in the variations of the tracer concentrations in the potential source flows would tend to diminish the significance of errors associated with reading concentration values from the instrument that are lower than the PQL.

A quick (and conservative) estimate of the needed detection limit can be made by only knowing the median concentration and the concentration variation of the tracer in the least contaminated component flow. Any amount of another component having a greater tracer concentration will increase the tracer concentration of the mixture. By ignoring this increase, minimum detection limits can be estimated based on the numerous probability calculations presented in the background demonstration project report (Pitt and Lalor publication pending):

R0022900

COV value:	Multiplier for detection limit:
<0.5 (low)	0.8
0.5 to 1.25 (medium)	0.23
>1.25 (high)	0.12

As an example, if the baseflow tracer has a low COV (<0.5), then the estimated required detection limit is about 0.8 times the median tracer concentration.

More than 80 percent of the library categories (source flows and tracers) examined in Birmingham, Alabama during the demonstration of these procedures (shown on Table 5) had low COV values. About 15 percent had medium COV values, and about 5 percent had high COV values. Free available chlorine had medium or high COV values for almost all source categories. This is a major reason why chlorine is not used quantitatively to identify source flow components in outfall samples. Chlorine is used in a similar manner as an aesthetic parameter (e.g., turbidity or odor). If high chlorine concentrations are found at the outfall (greater than about 0.5 mg/L), then a major treated potable water leak is likely associated with the dry-weather flow.

Table 6 lists the detection limit requirements for the tracer parameter concentrations found during the Birmingham, Alabama, demonstration project. The recommended analytical methods satisfy most of the required detection limits, except for ammonia and surfactants in spring water and surfactants in potable water. The spring water ammonia concentrations were about equal to the detection limit, but because the variation in the ammonia concentrations were so large, a much lower detection limit would be preferable.

Figures 4 through 7 are probability plots showing the required analytical detection limits for mixtures of two source area flows both having low COV values (similar to the majority of expected conditions). Pitt and Lalor (publication pending) present similar plots for all possible combinations of COV values. These figures show four curves corresponding to four mixtures. PER100 is for a 100 percent solution of the flow having the higher tracer concentration, PER50 is for a solution having 50 percent each of two components, PER15 is for a solution of 15 percent of the component having the higher tracer concentration and 85 percent of the component having the lower tracer concentration, while PER0 is a solution only made of the component having the lower tracer concentration. Figure 4 is for two components that have mean concentrations differing by 1.33 times, Figure 5 is for a mixture where the component mean concentrations differ by five times, Figure 6 is for two components with mean concentrations differing by 20 times, and Figure 7 is for two components with mean concentrations differing by 75 times. Each figure shows the detection limits, relative to the lower base concentrations, for different probability of detection values. The detection limits required are reduced significantly as the means of the tracer components differ by greater amounts, especially for low probabilities of detection.

For example, if the two tracer mean concentrations vary by about five times (e.g., treated potable water and sanitary wastewater potassium concentrations from Table 5) and a mixture of 15 percent sanitary wastewater and 85 percent potable water needs to be identified with a 90 percent probability of detection, the required detection limit would be about:

1.4 [factor from Fig.5] x 1.6mg/l [potassium in treated potable water Table 5] = 2.2 mg/L

The more conservative approach stated above would result in a minimum detection limit of:

0.8 [factor for COV < 0.5] x 1.6mg/l = 1.2 mg/L.

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TABLE 6. DETECTION LIMIT REQUIREMENTS FOR TRACER CONCENTRATIONS FOUND IN BIRMINGHAM, ALABAMA WATERS

Tracer Parameter and Units	Median Conc. (mg/L) of Least Contaminated Sources: median (COV)	Required Detection Limit	Available Detection limit[1]
Fluorescence % of full scale	Potable water: 4.6 (0.08) Spring water: 6.8 (0.43)	3.7 5.4	0.1
Potassium mg/L	Spring water: 0.73 (0.10) Potable water: 1.6 (0.04)	0.58 1.3	0.01
Ammonia mg/L	Spring water: 0.01 (1.7) Potable and Radiator water: 0.03 (0.23)	0.001 0.024	0.01
Fluoride mg/L	Spring water: 0.031 (0.87) Sanitary wastewater: 0.77 (0.23)	0.01 0.62	0.01
Surfactants mg/L as MBAS	Spring and potable water: <1 Sanitary wastewater: 1.5 (0.82)	- 0.35	0.01
Hardness mg/L as $CaCO_3$	Laundry water: 14 (0.57) Potable and radiator water: 49 (0.03)	3.2 39	1
Color HACH™ color units	Spring and potable water: <1 Sanitary wastewater: 38 (0.55)	- 8.7	1
Specific Conductivity µS/cm	Potable water: 110 (0.01) Spring water: 300 (0.04)	88 240	10

(1) From anlaytical methods discussed under: "Recommended Analytical Methodology".

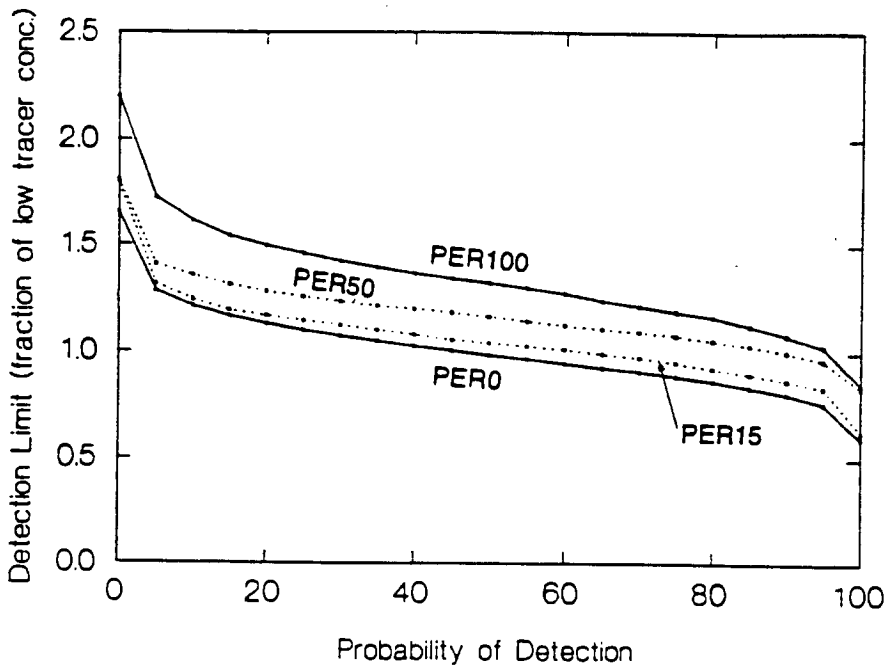

Figure 4. Required detection limits for low COV mixture components having means differing by 1.3 times.

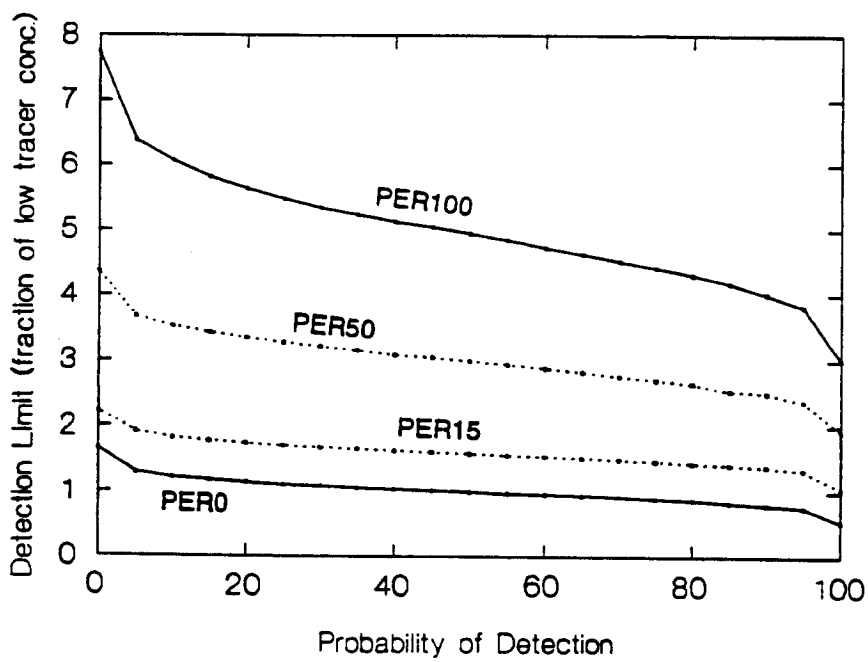

Figure 5. Required detection limits for low COV mixture components having means differing by 5 times.

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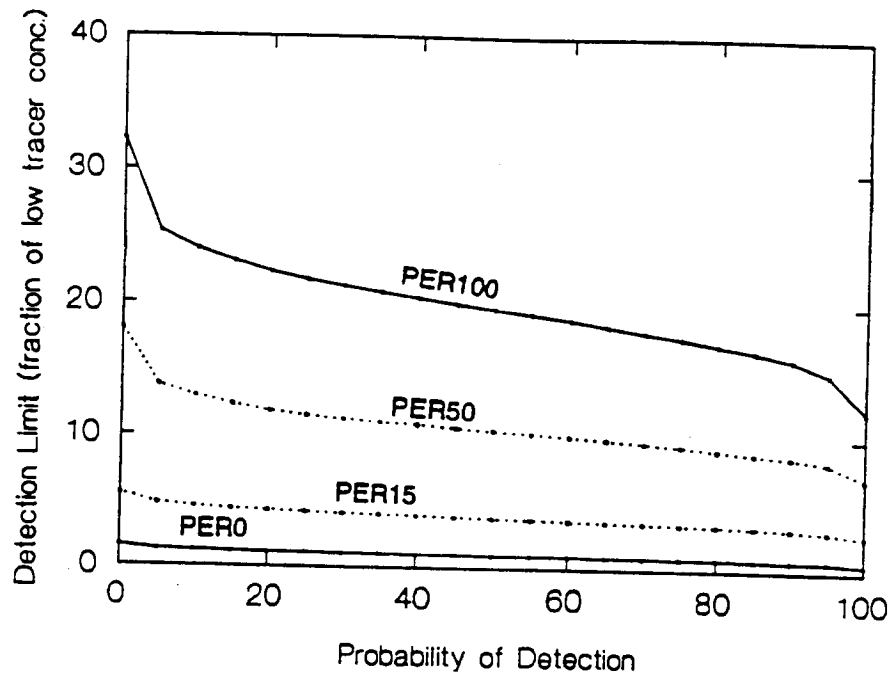

Figure 6. Required detection limits for low COV mixture components having means differing by 20 times.

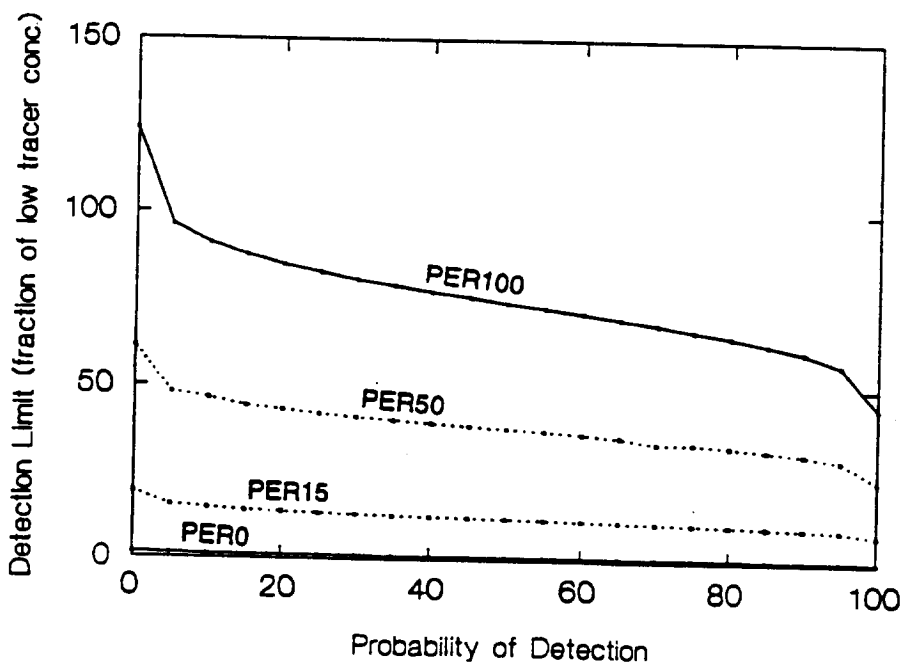

Figure 7. Required detection limits for low COV mixture components having means differing by 75 times.

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Even with the above analytical requirements satisfied, it may still be difficult to precisely estimate the degree of contamination, especially for low contamination levels and for high COVs. The ratio of the tracer concentration in the contaminating source flow to the tracer concentration in the cleaner baseflow must increase as the desire to detect smaller contaminating source flows is required. Listed below, for 90 percent confidence levels and low COV values, are percentages of source flow in the baseflow and the corresponding minimum concentration ratios (source to clean baseflow tracer concentrations) required for the detection of the source flow contamination of the baseflow.

Percent of Source Flow Contamination in Baseflow:	Required concentration ratios (low COV values):
1%	50
5%	10
10%	7
25%	3
35%	1.5
50%	1.2

As an example, the median tracer concentration in the contaminating source flow must be about 10 times greater than the median tracer concentration in the cleaner baseflow to detect a five percent source flow contamination of the baseflow. If the tracer COV values are "medium" or "high", then the required concentration differences are much greater (up to 250 times difference in concentrations may be required).

Therefore, the differences in tracer concentrations must be quite large, and the COVs quite small, in order to have confident estimates of low levels (percentages) of contaminating source flows. Few tracers exhibit such a wide range in characteristics between source flow and baseflow categories. This is the main reason why the use of multiple tracers for source flow identification is important. Some tracers may not uniformly produce good estimates of contaminating source flow levels, but the use of redundant tracers for the same decision (e.g., ammonia and potassium to identify sanitary wastewater; fluorides and hardness to identify treated potable water; and surfactants and fluorescence to identify wash waters) and good estimates of local contaminant characteristics, will minimize these errors.

The actual minimum level of contaminating source flow that will be detectable will be dependent on the analytical precision, as discussed next.

Required Sample Analytical Precision

The repeatability of the analytical method is an important consideration in its selection. Precision, as defined in Standard Methods (APHA, et al. 1989), is a measure of the closeness with which multiple analyses of a given sample agree with each other. It is determined by repeated analyses of a stable standard, conducting replicate analyses on the samples, or by analyzing known standard additions to samples. Precision is expressed as the standard deviation of the multiple analysis results.

Figure 8 is a summary of the probability plots from Pitt and Lalor (publication pending) and indicates the needed analytical precision (repeatability) as a fraction of the median tracer concentration (i.e., the flow with the lower tracer concentration) to resolve one percent contamination of the baseflow by the source flow, at a 90 percent confidence level. This figure was developed for COV values of the tracer parameters in the contaminating flows ranging from 0.16 to 1.67.

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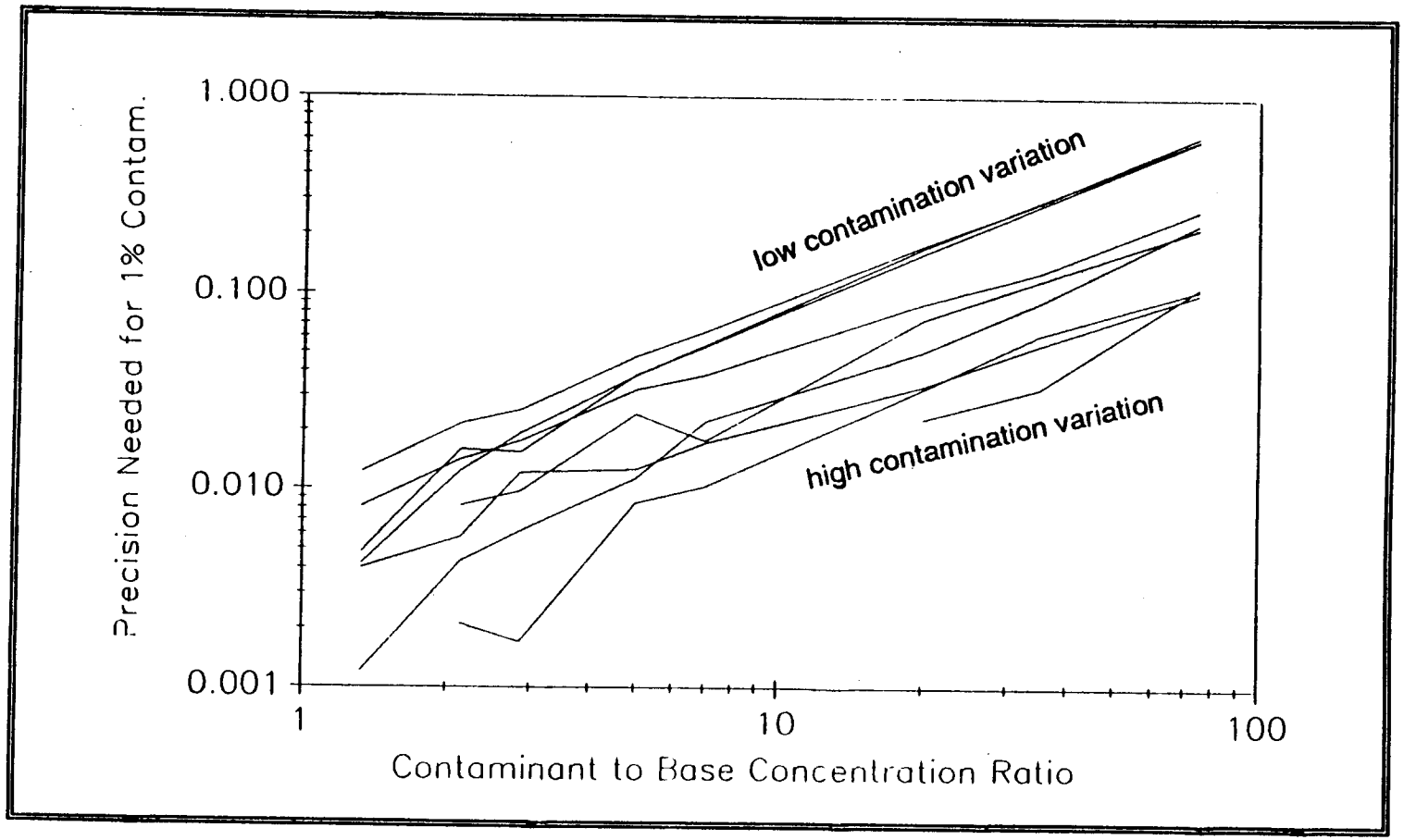

Figure 8. Analysis precision needed for detection of one percent contamination at ninety percent confidence.

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If the available analytical precision is worse than these required values, then small contaminating flow levels may not be detected. Therefore, even with adequate analytical detection limits, poor analytical precision may not allow adequate identification of low levels of contaminating flow. In many cases, it is expected that a contaminating flow level of just a few percent can cause significant toxic and pathogenic problems. Examples include gasoline spills, direct connections of raw sanitary wastewater, and metal plating bath wastewaters.

If the tracer concentrations of the flow components are close in value and the variation of the concentrations are high, then it will be very difficult to adequately discern flow components. In contrast, if the tracer concentrations of the flow components are widely different and have low variabilities, then much smaller levels of contaminating flows could be detected. As an example, if the median contaminant tracer concentrations differ by a factor of 10 in two flow components, but have high concentration variations (high COV values), a precision of between 0.015 to 0.03 of the lower baseflow median tracer concentration is needed, for each percent of contaminating flow that needs to be detected. If the median tracer concentration in the cleaner baseflow is 0.15 mg/L (with a corresponding tracer median concentration of 10 times this amount, or 1.5 mg/L, in the contaminating source flow), then the required analytical precision is about 0.015 x 0.15 = 0.002 mg/L to 0.03 x 0.15 = 0.005 mg/L per one percent of contaminating flow to be detected. If at least five percent of contaminating flow is needed to be detected, then the minimum precision would have to be 5 x 0.002 = 0.01 mg/L.

The conservative method noted previously can be used to estimate the detection limit requirements for the above example:

low COV in the cleaner baseflow: 0.8 x 0.15 mg/L = 0.12 mg/L
medium COV in the cleaner baseflow: 0.23 x 0.15 mg/L = 0.035 mg/L
high COV in the cleaner baseflow: 0.12 x 0.15 mg/L = 0.018 mg/L.

The required analytical precision would therefore be about one-half of the lowest detection limit needed, and about 1/12 of the largest estimated required detection limit.

Recommended Analytical Methodology

An important part of the development of these investigation procedures and the demonstration project (Pitt and Lalor publication pending) was the laboratory and field testing of alternative analytical methods. Dry-weather outfall samples were subjected to different tests which compared several analytical methods for each of the major tracer parameters of interest. Tests were conducted to enable comparison of the results of alternative tests with standard procedures and to identify which methods had suitable detection limits, based on real samples. In addition, representative samples were further examined using standard addition methods (known amounts of standards added to the sample and results compared to unaltered samples) in order to identify matrix interferences. Matrix interferences are generally caused by contaminants in the samples interfering with the analysis of interest. Many of the analysis methods were also tested against a series of standard solutions to identify analytical precision (repeatability), linearity, and detection limits. The following paragraphs (and Table 7) summarize the recommended analytical procedures.

Most of the recommended analyses are conducted using small "field-type" instruments. However, despite their portability, the use of these instruments in the field can introduce many errors. Temperature and specific conductivity are the only analyses that are recommended for field analyses. For the other analyses, samples are collected at the site, iced, and taken back to the laboratory for analyses. The recommended analytical procedures can be easily conducted in a temporary laboratory; all that is needed is a work space and adequate ventilation. Access to power and water would be

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TABLE 7. SAMPLE ANALYSES LAB SHEET

Sample number:_____

Date:_____

Location:_____

Outfall #:_____

Specific conductivity YSI™ SCT meter (field)_____

Temperature YSI™ SCT meter (field)_____

pH pH meter (lab)_____

Ammonia Direct Nesslerization (lab)_____

Color HACH™ color kit (lab)_____

Fluoride HACH DR/2000™ spect. with AccuVacs™ (lab)_____

Hardness HACH™ field titration kit (lab)_____

Surfactants HACH™ detergent field kit (lab)_____

Fluorescence Turner™ fluorometer (lab)_____

Potassium HACH DR/2000™ spect. (lab)_____

Turbidity HACH™ Nephelometer (lab)_____

Chlorine HACH DR/2000™ spect. with AccuVacs™ (lab)_____

Toxicity Microtox™ 100% sample screen (lab)_____

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helpful, but all of the equipment can be operated with batteries. At each outfall, a (2 L) sample of dry-weather discharge needs to be collected and stored in a polyethylene container. Another (500 mL) sample can also be collected in a glass container having a Teflon-lined lid for toxicity screening and selected toxicant analyses. All samples must be analyzed (or extracted) within accepted time limits.

Descriptions of the procedures and parameters recommended for the analysis and identification of dry-weather outfall samples are:

Water color--

Determine in the laboratory using a simple comparative colormetric (color wheel) field test kit from the HACH Company. Apparent color (unfiltered samples), expressed in HACH color units.

pH--

pH is measured in the laboratory using a standard laboratory pH meter after accurate calibration using at least two buffer solutions bracketing the expected sample pH value. (pH measurements using pH test paper have been found to be generally within one unit of the laboratory meter. However, this difference is too large and is not recommended. Small "pen" pH meters most suitable for field use can easily be off by a 0.5 pH unit and are relatively hard to calibrate. They accordingly must be used with care.)

Specific conductivity and temperature--

These parameters are quickly and easily measured in the field using a multi-parameter SCT meter from YSI model 33. Both specific conductivity and temperature must be calibrated against standard specific conductivity solutions and a standard thermometer. Specific conductivity should also be corrected to standard values obtained at 25°C (APHA, et al. 1989):

$$K = (K_m C)/[1 + 0.0191(t-25)]$$

where K = specific conductivity at 25°C

K_m = measured specific conductivity at temperature t°C

and C = cell constant

The cell constant is a correction factor determined by measuring a 0.01M KCl solution at 25°C, after three rinses, compared to 1413 μS/cm, the expected value. This equation results in about a 2% change in specific conductivity for every degree in temperature difference from 25°C. The International System of Units (Système International d' Unités, SI) specific conductivity unit of measurement is the μS/cm which is numerically equivalent to the U.S. Customary unit, μmhos/cm.

Fluoride--

Easily analyzed in the laboratory using a field spectrophotometer and evacuated reagent and sample vessels (HACH DR/2000™ and AccuVac™ ampules using SPADNS reagent, without distillation). The AccuVac™ procedure works well for sample concentrations less than 2.5 mg/L; however, in rare instances of higher concentrations, sample dilution is required because of non-linear instrument responses. The samples should be filtered through a 0.45 μ membrane filter (e.g., Millipore™ filter) before analysis to minimize color interference. (Specific-ion probes were also evaluated, but the technique proved to be too inconsistent, especially for personnel having little training.)

R0022909

Ammonia--

Easily measured in the laboratory using a direct Nesslerization procedure and spectrophotometer (HACH DR/2000™ Nessler method, but without sample distillation). The samples should be filtered through a 0.45 μ membrane filter before analysis to minimize color interference. (The use of various indicator test papers and simple field test kits for ammonia determination gave poor results. Specific-ion probes were also tested. Typical problems encountered for these procedures, (except for the direct Nesslerization procedure), were color interferences, long analysis times, inconsistent results, and poor performance when standard solutions were analyzed.)

Potassium--

Measured in the laboratory either using a spectrophotometer (HACH DR/2000™ Tetraphenylborate method), or a flame atomic absorption spectrophotometer (if available). The samples should be filtered through a 0.45 μ membrane filter before spectrophotometric analysis to minimize color interference. (Specific-ion probes were also evaluated and indicated the same poor results found for fluorides and ammonia.)

Surfactants--

Measured in the laboratory using a simple comparative colormetric (color wheel) method (from the HACH Company). The samples should be filtered through a 0.45 μ membrane filter before analysis to minimize color interference. This procedure should be carried out under a laboratory fume hood. (Specific-ion probe titrations for surfactants were not successful because of poor detection limits.)

Fluorescence--

Analyzed using a laboratory fluorometer (Turner model 111). The fluorometer had general purpose filters and lamps and was operated at the most sensitive setting (number one aperture).

Hardness--

Determined in the laboratory using a field-titrimetric kit (HACH Digital Titrator Model 16900). The samples should be filtered through a 0.45 μ membrane filter before analysis to minimize color interference. (A number of simple field test kits were tested but the direct reading titration method proved most convenient and accurate. However, hardness test paper can be used to estimate the titration end point.)

Turbidity--

Determined using a HACH Nephelometer in the laboratory.

Chlorine--

Total available chlorine was determined with the DPD (N, N-diethyl-p-phenylenediamine) method using a HACH DR/2000™ spectrometer with AccuVac™ ampules.

Toxicity-screening--

Toxicity screening tests have been found to be very useful as indicators of contamination of storm drains. The Microtox™ (from Microbics) toxicity screening test can be used for relative toxicity values. The 100 percent screening test was most commonly used. If the light output decrease after 25 minutes (the I_{25} value) was greater than 50 percent, then the standard Microtox test was used to determine the sample dilution required for a 50 percent light decrease (the EC50 value). If a sample results in a large toxic response, then specific toxicant analyses (organics and metals) could be performed to better identify the toxicant source. In general, the Microtox™ screening test was found to be an efficient method for toxicity analysis, particularly for identifying samples requiring further analyses. (A number of simple test kits were used for specific heavy metal analyses, but with very poor results. High-detection limits and interferences make these methods impractical, unless an outfall is grossly contaminated with a concentrated source, such as raw plating bath wastewater.)

R0022910

SECTION 5

INITIAL FIELD SCREENING SAMPLING ACTIVITIES

SAMPLING STRATEGY

The importance of sampling all outfalls, regardless of size, should be stressed. Figure 9 shows the distribution of outfalls for the Birmingham, Alabama area surveyed for the city's stormwater discharge permit application. The median equivalent diameter of the 566 outfalls that had drainage area estimates available was 36 in. About 20 percent of the outfalls were greater than 60 in. in diameter and about 20 percent were less than 20 in. in diameter. Most of the largest outfalls were actually drainage ditches. There was an average of about 70 acres draining to each outfall, but the drainage areas ranged from much less than one acre to over 1500 acres. About 40 percent of the outfalls were affected by either commercial or industrial land uses and would therefore be considered as critical drainage areas for both dry-weather flows and stormwater runoff.

The Birmingham, Alabama demonstration project that tested this protocol covered a residential and commercial drainage area having approx. 70 outfalls. The median outfall size of the outfalls in this study area was 16 in., and more than 75 percent of the outfalls were less than 36 in. in diameter. Examination of the outfalls during seven separate sampling occasions found that while some of the dry-weather flows occurred intermittently, most were continuous. About 25 percent of the outfalls were found to be consistently flowing during dry weather, with about two-thirds of the flows discharging from pipes that were less than 36 in. in diameter. About five percent of the outfalls exhibited dry-weather flows which were extremely toxic or were raw, undiluted, sanitary wastewater. Each of these contaminated outfalls were 20 in., or less, in diameter. Some of the worst dry-weather flow discharge problems were associated with very small (4 in. diameter) pipes draining automobile service areas adjacent to the receiving water. It was found that small outfalls can contribute significant pollutant loads to receiving waters and should not be neglected if receiving water improvement is a serious goal.

FIELD DATA COLLECTION

Before the field data can be collected, preliminary mapping and land use evaluation work is needed. Section 3 described the preliminary work and the likely data sources for the information that is needed before the field investigations can begin. The most important preliminary information required is:

- outfall locations,
- outfall drainage areas,
- commercial and industrial activities in each drainage area, and
- locations of septic tanks in the individual drainage areas.

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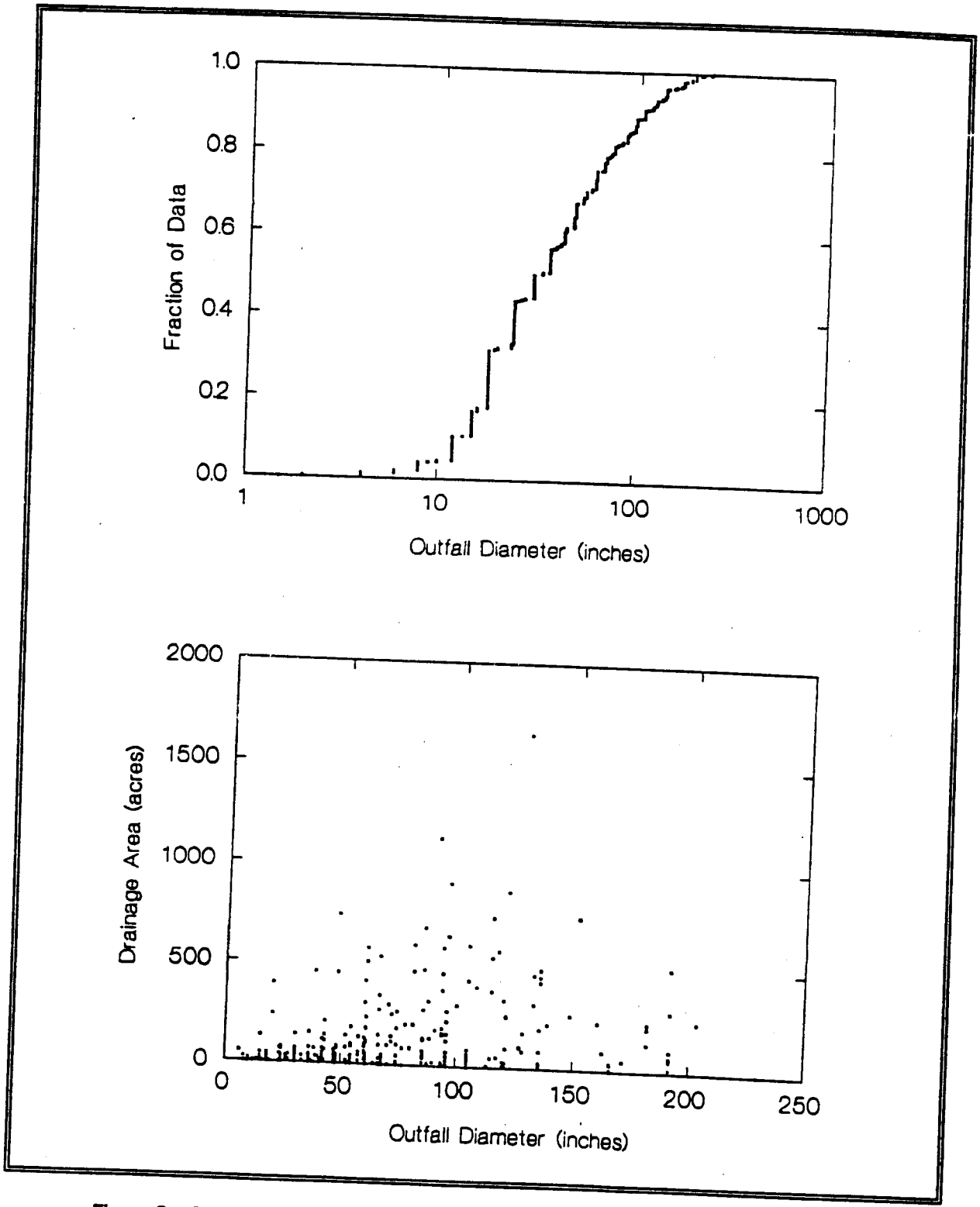

Figure 9. Outfall characteristics for Birmingham, Alabama demonstration project.

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Outfall Locations

Frequently, city maps of known outfall locations are inadequate. Many outfalls are not located on city drainage maps because of infrequent or improper updating, or unauthorized installations. Because it is very difficult for communities to maintain up-to-date maps of drainage facilities, actual stream surveys are needed to verify and update existing information. Illicit outfalls will not usually be shown on maps, and field surveys will be required to detect these as well. Most newer developments do have accurate drainage and outfall maps, but the outfall locations may not have been transferred to an overall city map. A few cities have Geographic Information Systems (GIS) in place and are including the storm drainage systems on appropriate data overlays. It is important to identify all outfalls because present data indicates no relationship between the most significant sources of non-stormwater discharges and the largest drainage areas, or the largest diameter outfalls.

Because of the likelihood of poor data concerning the outfall locations, it will probably be necessary to "walk" the creeks and actively look for outfalls. In most cases, it requires several trips (about three) to locate all outfalls. The initial outfall surveys should be conducted during times when riparian vegetation is minimal. Whenever an outfall is located, it needs to be marked (coded using spray paint or by other means).

If the receiving water is a small creek, it can be waded in a downstream direction. If the receiving water body cannot be waded, a small boat or canoe can be used to look for outfalls above the water. Submerged outfalls are more difficult to find and require more careful inspections for storm drain manholes along the shore. In flood or estuary tidal areas, surveys should be conducted during low tides when more outfalls are likely to be exposed. In many cities, streets parallel the banks of creeks or drainage canals that contain outfalls. It may be possible to carefully search the opposite bank from a moving automobile. It may also be cost-effective to use light aircraft (including helicopters) to search for outfalls. Submerged outfalls could be easier to identify from the air than from the water in cases where discharge plumes are visible.

Obviously, outfall characterizations should be conducted during these surveys, if possible. In all cases, at least two people are needed to look for outfalls, especially if wading a creek. Another person can drive a shuttle car to a convenient downstream location for crew rotation.

Field Survey

The main elements of the field sampling plan are the collection of necessary information and equipment, and preliminary screening of outfalls.

Collect necessary information and equipment--

Maps--Maps are the most important part of the field equipment. Adequate field maps can be prepared by enlarging standard USGS 7-1/2 minute quadrangle maps to appropriate scales. In addition, detailed street maps are also needed to locate specific street crossings and to identify locations of outfalls in the field.

Field sampling and analysis equipment--Table 8 lists the equipment that is needed for a field survey. In no case should personnel conduct the field surveys alone, wade streams without wearing waders, or be in boats without wearing life preservers. Heavy duty waders (heavy Cordura™ nylon) are preferred. Urban streams contain appreciable debris (broken bottles, etc.). In addition, urban

R0022913

TABLE 8. FIELD EQUIPMENT LIST

Temperature and specific conductivity meter.

Field notebook containing maps and non-stormwater flow evaluation field sheets.

Waterproof marker/pen.

Camera and film.

Spray paint.

Tape measures (both 3m and 30m).

Flashlight.

Watch (with second hand).

Glass sample containers with waterproof labels (500 mL).

Plastic sample containers with waterproof labels (1 to 2 L.).

Ice boxes with ice (left in vehicle).

Backpack.

Grab water sampler (dipper on long pole).

Hand operated vacuum pump sampler for shallow flows.

Waders and walking stick.

First aid kit and pocket knife.

Self protection pepper spray.

Two-way radios for communication between field crew and van driver.

Hand held GPS (global positioning satellite) system receiver (only capable of locating positions within about 100 to 350 feet).

R0022914

streams are isolated wildlife areas which tend to concentrate certain wildlife species that live in close proximity to man (including cottonmouths, water moccasins, copperheads, and rattlesnakes), plus contain lush growths of poison ivy or oak. The self protection pepper spray may be especially handy in case of harassing dogs.

This equipment would supplement needed boating equipment, if boats are used. Some of this equipment (ice coolers and ice, along with extra bottles) would be kept in the vehicle. In most cases, the vehicle should be moved in about 1/2 mile increments. This length would typically contain up to ten outfalls, with relatively few flowing outfalls to sample. The collected samples would therefore be iced within about 1/2 hour of collection. It is possible that the vehicle driver could conduct critical analyses (chlorine, pH and ammonia) while waiting. It is suggested that a three person crew rotate, with a new driver at each new shuttle location.

Arrange for lab testing and other support equipment--Before the field crew goes into the field to collect samples, the laboratory needs to be notified and ready to analyze the samples soon after they are available. As shown in the next section, the laboratory testing procedures for the basic tracer parameters are all simple and can be conducted in an unsophisticated laboratory. It may be feasible for the field crew to conduct the sample analyses in the afternoon of the day when they are collected.

Preliminary screening of outfalls--

Location of outfalls--Outfall locations need to be transferred to field maps and the daily activities planned. The number of outfalls that can be visited and sampled in a single day is highly dependent on outfall accessibility and mobility along the receiving water. The initial survey requires the longest time, after which repeated surveys require much less effort. In a small creek having shallow and slow water with numerous road crossings, about three miles of creek can be walked (with about 40 outfalls visited and ten outfall samples obtained) in a half-day of field activity with a crew of three people. Most other conditions would require additional labor for the same sampling effort. In all cases, careful planning, especially having an idea of where the outfalls are located, would greatly reduce the labor involved.

Scheduling field surveys--It is important to schedule the field surveys during low water levels (during low tides or low flows) because outfalls could be submerged and concealed during high water conditions. It is also best not to conduct the field surveys during periods of high flow in the receiving waters because of safety concerns.

Field surveys which are timed (diurnally, or seasonally) to coincide with periods with a greater potential for non-stormwater entries, are likely to reveal more dry-weather discharges. As examples, morning periods (or in areas of tourism, during the tourist season) usually experience the greatest sanitary wastewater flows. Scheduling sampling during these morning hours would be most successful in identifying sanitary wastewater contamination of the storm drainage system. Many inappropriate industrial entries to the storm drainage system also occur on a scheduled basis, e.g., cleaning up work areas between work shifts, or increased wastewater flows during periods of the year when the specific industry is especially busy. Again, investigating potentially affected storm drain outfalls during these critical periods would result in better data.

The field survey schedule will need to be flexible to avoid sampling during and immediately after a storm event, to ensure only dry-weather flows are recorded. In most urban areas storm runoff drainage flows will cease within 12 hours following the storm event, but this will need to be reviewed for each watershed area. The time to flow through the upstream drainage system and any detention and subsequent release of the storm water could extend this 12 hour period. This subject is discussed

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further under Section 5, Irregular Flows.

Sampling techniques--After an outfall is located, it is labeled with paint or marked by other means and the form shown on Table 9 is completed in the field. Table 10 describes the physical observation choices, previously discussed in Section 4. The use of field sheets and laboratory record keeping is very important because of the large number of outfalls that will likely be surveyed in each municipality.

Table 9 is a field sheet that can be used to record the observations and analytical results for the outfall survey. The top of the sheet includes basic outfall descriptive and weather information, a flow rate estimate, and an indication if industrial or commercial activities are known to occur in the area. The physical observation data section requires simple circling of the most appropriate value, or writing in another response. Samples should be obtained of floatable and staining materials for further laboratory microscopic analyses. If unusual vegetative conditions or damage to structures are found, then the extent and appearance of the damage should be described. In all cases, several photographs need to be taken of outfall conditions for each site visit. The analyses results are written on the form, along with a short descriptions of the equipment used.

Flows are estimated and visually characterized for each outfall visit. Field temperature and specific conductivity measurements are made in the field, and dry-weather discharge water samples are collected for later (same day) laboratory analyses. A single water sample (1 to 2 L) is sufficient for almost all analyses that may be conducted on the sample. This sample can be collected in a polyethylene collapsible container. In addition, another (500 mL) sample can be collected in a glass bottle (having a Teflon lined lid) if a toxicity screening procedure (like Microtox™) and selected organic tracers are to be analyzed. Specific sample volume requirements need to be determined in conjunction with the laboratory personnel. Excess samples should be placed in smaller polyethylene bottles and frozen for potential future analyses (e.g., heavy metals and major ions).

Sample preservation--Usually icing of samples after collection and same-day laboratory analyses is adequate. Ammonia, chlorine, and pH are susceptible to change with time and special tests may be needed to determine the tolerable delay before laboratory analyses. As noted previously, it is not efficient to analyze the samples in the field, especially after each sample is collected.

Field tests--The only tests recommended for field analyses are temperature and specific conductivity. If a multi-purpose temperature/specific conductivity meter is being used for the temperature analyses, then both can be easily determined in the field.

Record keeping, sample preservation, and analyses--As noted above, the collected water samples need to be analyzed soon after collection. A central laboratory is much more effective than trying to analyze each sample in the field as it is collected. Section 4 presents the recommended laboratory procedures.

Data analyses--

Identification of contaminated outfalls--Section 6 describes several methods to identify the likely components in each flowing outfall. This information is then used to identify the contaminated dry-weather flows.

Isolation and correction of contaminating flow sources--After the problem outfalls are identified, drainage system surveys are used to find the sources of the contaminating flows. These procedures are briefly discussed later in this User's Guide.

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TABLE 9. SAMPLE EVALUATION SHEET

Outfall # _____ Photograph # _____ Date: _____

Location: _____

Weather: air temp.: ___ °C rain: Y N sunny cloudy

Outfall flow rate estimate: ____ L/sec

Known industrial or commercial uses in drainage area? Y N
describe: _____

PHYSICAL OBSERVATIONS:

Odor: none sewage sulfide oil gas rancid-sour other:_____

Color: none yellow brown green red gray other:_____

Turbidity: none cloudy opaque

Floatables: none petroleum sheen sewage other:_____ (collect sample)

Deposits/stains: none sediment oily describe:_____ (collect sample)

Vegetation conditions: normal excessive growth inhibited growth
extent: _____

Damage to outfall structures:
identify structure: _____
damage: none / concrete cracking / concrete spalling / peeling paint / metal corrosion
other damage: _____
extent: _____

ANALYSES:		EQUIPMENT USED:
Specific conductivity:	___ µS/cm	_____
Temperature:	___ °C	_____
Fluoride:	___ mg/L	_____
Hardness:	___ mg/L	_____
Surfactants:	___ mg/L	_____
Florescence:	___ % of scale	_____
Potassium:	___ mg/L	_____
Ammonia:	___ mg/L as N	_____
pH:	___	_____

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TABLE 10. INTERPRETATIONS OF PHYSICAL OBSERVATION PARAMETERS AND LIKELY ASSOCIATED FLOW SOURCES

Odor - Most strong odors, especially gasoline, oils, and solvents, are likely associated with high responses to the toxicity screening test. Typical obvious odors include: gasoline, oil, sanitary wastewater, industrial chemicals, decomposing organic wastes, etc.

sewage: smell associated with stale sanitary wastewater, especially in pools near outfall.
sulfide ("rotten eggs"): industries, e.g., meat packers, canneries, dairies, etc; and stale sanitary wastewater.
oil and gas: petroleum refineries or facilities associated with vehicle maintenance and operation or petroleum product storage.
rancid-sour: food preparation facilities (restaurants, hotels, etc.).

Color - Important indicator of inappropriate industrial sources. Industrial dry-weather discharges may be of various colors, but dark colors, such as brown, gray, or black, are most common.

yellow: chemical, textile, and tanning plants.
brown: meat packers, printing plants, metal works, stone and concrete works, fertilizer application, and petroleum refining facilities.
green: chemical plants, and textile facilities.
red: meat packers.
gray: dairies.

Turbidity - Often affected by the degree of gross contamination. Dry-weather industrial flows with moderate turbidity can be cloudy, while highly turbid flows can be opaque. High turbidity is often a characteristic of undiluted dry-weather industrial discharges.

cloudy: sanitary wastewater, concrete or stone operations, fertilizer facilities, and automotive dealers.
opaque: food processors, lumber mills, metal operations, and pigment plants.

Floatable Matter - A contaminated flow may contain floating solids or liquids directly related to industrial or sanitary wastewater pollution. Floatables of industrial origin may include animal fats, spoiled food, oils, solvents, sawdust, foams, packing materials, or fuel.

oil sheen: petroleum refineries or storage facilities and vehicle service facilities.
sewage: sanitary wastewater.

(continued)

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TABLE 10. (continued)

Deposits and Stains - Refer to any type of coating near the outfall and are usually of a dark color. Deposits and stains often will contain fragments of floatable substances. These situations are illustrated by the grayish-black deposits that contain fragments of animal flesh and hair which often are produced by leather tanneries, or the white crystalline powder which commonly coats outfalls due to nitrogenous fertilizer wastes.

sediment: construction site erosion.
oily: petroleum refineries or storage facilities and vehicle service facilities.

Vegetation - Vegetation surrounding an outfall may show the effects of industrial pollutants. Decaying organic materials coming from various food product wastes would cause an increase in plant life, while the discharge of chemical dyes and inorganic pigments from textile mills could noticeably decrease vegetation. It is important not to confuse the adverse scouring effects of high stormwater flows on vegetation with highly toxic dry-weather intermittent flows.

excessive growth: food product facilities.
inhibited growth: high stormwater flows, beverage facilities, printing plants, metal product facilities, drug manufacturing, petroleum facilities, vehicle service facilities and automobile dealers.

Damage to Outfall Structures - Another readily visible indication of industrial contamination. Cracking, deterioration, and spalling of concrete or peeling of surface paint, occurring at an outfall are usually caused by severely contaminated discharges, usually of industrial origin. These contaminants are usually very acidic or basic in nature. Primary metal industries have a strong potential for causing outfall structural damage because their batch dumps are highly acidic. Poor construction, hydraulic scour, and old age may also adversely affect the condition of the outfall structure which are not indications of upstream contaminating entries.

concrete cracking: industrial flows
concrete spalling: industrial flows
peeling paint: industrial flows
metal corrosion: industrial flows

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Irregular Flows

Irregular flows pose a special problem during the field surveys. Outfall apparent "dry-weather" flows can be intermittent in nature, only flowing soon after rains and then remaining dry, or may flow when inappropriate water sources enter the storm drainage system. If irregular flows are associated with rains, outfall surveys should be postponed until sufficient time has lapsed since the last major rain. For most urban areas, storm runoff drainage ends several hours (but usually less than 12) after the rain stops. Extended, but decreasing flows, after rains could be associated with high groundwater or percolating rain water infiltrating into the drainage system. In this case, most outfall surveys should be further delayed. However, some pollutant sources may be associated with these after storm flows, especially contaminated groundwaters (septic tank problems, leaky underground storage tanks, etc.). Therefore, it may be important to sample these flows, especially if these contaminant sources potentially exist.

Basic field indicators, such as the presence of residual stains or deposits, oil sheens, coarse solids, floatables, color, odors, etc., in the absence of a flow, indicate the likelihood of intermittent dry-weather flows. These observations will be enhanced by installing simple "tell-tale" devices, e.g., a terry-cloth (strain the discharge) or small caulk dam in the drain. Outfalls exhibiting these signs of non-continuous discharges should be visited several times to increase the probability of observing and sampling a dry-weather discharge. Analyzing pooled water immediately below the outfall or collected between visits in small, constructed dams within the storm drain can greatly assist in identifying non-continuous discharges. Coarse solids and/or floatables can be captured through the erection of coarse screens and/or booms at a manhole site, the mouth of the outfall, or in the receiving stream. It may be necessary to visit suspect outfalls frequently. However, it is virtually impossible to capture an isolated short-term intermittent flow (e.g., from the illegal dumping of wastes into the storm drainage system) from outfall visits.

Simple outfall area characteristics, noted above, are the most reliable indicator of a potential intermittent source at an outfall. In addition to using a dam, or other indicator device (e.g., a small screen to capture particulate debris), it may be desirable to use an automatic water sampler at especially important outfalls. Automatic samplers would be unreasonable and expensive to use at many outfalls in an area and test locations would need to be carefully selected. A sampler located in a close-by manhole and set to sample every fifteen minutes (with four samples placed in each bottle) can monitor for intermittent flows for a period of 24 hours. Automatic samplers can also be used to characterize variable quality flows. This information can be valuable in identifying possible discharge sources.

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SECTION 6

DATA ANALYSIS TO IDENTIFY PROBLEM OUTFALLS AND FLOW COMPONENTS

The field screening surveys are to be used as an initial effort to identify the outfalls needing more detailed drainage area investigations which would identify specific pollutant sources and control options. These field screening surveys, discussed in Sections 4 and 5, include physical, chemical, and relative toxicity evaluations of outfall and/or discharge conditions.

The purpose of the procedures presented in this User's Guide is to separate storm drain outfalls into general categories (with a known level of confidence) and to identify which outfalls (and drainage areas) need further analyses and investigations. The categories used in this Guide are outfalls affected by non-stormwater entries from: (1) pathogenic or toxic pollutant sources, (2) nuisance and aquatic life threatening pollutant sources, and (3) unpolluted water sources.

The pathogenic and toxic pollutant source category should be considered the most severe because it could cause disease upon water contact or consumption and cause significant impacts on receiving water organisms. They may also cause significant water treatment problems for downstream consumers, especially if they contain soluble metal and organic toxicants. These pollutants may originate from sanitary, commercial, and industrial wastewater non-stormwater entries. Other important residential area activities that may also be considered in this most critical category (in addition to sanitary wastewater) include inappropriate household toxicant disposal, automobile engine de-greasing, vehicle accident clean-up, and irrigation runoff from landscaped areas excessively treated with chemicals (fertilizers and pesticides).

Nuisance and aquatic life threatening pollutant sources can originate from residential areas and can include laundry wastewater, landscaped area irrigation runoff, automobile washing, construction site dewatering, and washing of concrete mixing trucks. These pollutants can cause excessive algal growths, depressed dissolved oxygen concentrations, tastes and odors in downstream water supplies, offensive coarse solids and floatables, and highly colored, turbid or odorous waters.

Relatively clean or unpolluted water discharged through stormwater outfalls can originate from natural springs feeding urban creeks that have been converted to storm drains, infiltrating groundwater, and infiltrating potable water from water line leaks.

A method must be used to compare data from individual outfall dry-weather samples to the library of dry-weather source flow data to identify which outfalls belong in which general category of contamination listed above. This comparison should result, at the very least, in the identification of the outfalls that are considered as major pollutant sources for immediate remediation. The degree of detail which can be identified for an outfall will depend on the extent of the local data collected to describe the likely source flows.

The procedures that can be used to identify outfall flow components may begin with simple yes/no checks. For example, if no surfactants are measured in an outfall sample, then sanitary wastewater is unlikely to be a contributor to the outfall flow. If no fluoride is measured, then fluoride

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treated potable water sources could be ruled out as contributors. The probability that remaining contenders are present alone or in a mixture may be determined using a combination of matrix algebra and the selecting of random values from within specified ranges using a Monte Carlo process and many iterations.

Most contaminated outfalls will require correction before the receiving water quality recovers to acceptable levels. However, ranking the outfalls allows the most serious outfalls to be recognized and enables corrective action to be initially concentrated in the most cost-effective manner. In some of the case studies investigated, correcting only problems at the most critical outfalls resulted in insufficient receiving water quality improvements. It may be important to eventually correct all non-stormwater discharge problems throughout a city, not just the most severe problems. The field screening program should therefore be considered as an initial effort that needs to be followed-up with more detailed watershed drainage surveys in most of the areas having observed dry-weather flows. The follow-up watershed surveys are to identify and correct inappropriate pollutant entries into storm drainage systems, as discussed in Sections 7 and 8.

The identification of flow components of the dry-weather storm drain flow can be used to determine which outfalls have the greatest pollution potential. As an example, if an outfall contains sanitary wastewater, it could be a significant source of pathogenic microorganisms. Similarly, if an outfall contains plating bath water from a metal finisher, it could be a significant source of toxicants. These outfalls would be grouped into the most critical category of toxicants/pathogens. If an outfall contains washwaters from a commercial laundry or car wash, the wastewater could be a major source of nutrients and foaming material. These outfalls would be grouped into an intermediate category of nuisance and aquatic life threatening. Finally, if an outfall only contains unpolluted groundwater or water from leaky potable water mains, the water would be non-polluting and the outfall would be grouped into the last category of unpolluted water sources.

The five methods of data analyses presented in the following discussions present a hierarchy of methods, ranging from relatively simple reviews of the outfall characteristics to more sophisticated methods requiring computer modeling for evaluation. It is suggested that as many of the procedures be used as possible in evaluating the data, as each method provides some unique insights into the problems. Pitt and Lalor (publication pending) contains a more through discussion of these analysis procedures, including evaluation of the Birmingham, Alabama, demonstration project data.

INDICATORS OF CONTAMINATION

Indicators of contamination (negative indicators) are clearly apparent visual or physical parameters indicating obvious problems and are readily observable at the outfall during the field screening activities. These observations are very important during the field survey because they are the simplest method of identifying grossly contaminated dry-weather outfall flows. The direct examination of outfall characteristics for unusual conditions of flow, odor, color, turbidity, floatables, deposits/stains, vegetation conditions, and damage to drainage structures is therefore an important part of these investigations. Table 10 in Section 5 presented a summary of these indicators, along with narratives of the descriptors to be selected in the field.

This method does not allow quantifiable estimates of the flow components and if used alone will likely result in many incorrect determinations (missing outfalls that have important levels of contamination). These simple characteristics, discussed further below, are most useful for identifying gross contamination. Only the most significant outfalls and drainage areas would therefore be recognized from this method. The other methods, requiring chemical determinations, can be used to

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quantify the flow contributions and to identify the less obviously contaminated outfalls.

Indications of intermittent flows (especially stains or damage to the structure of the outfall) could indicate serious illegal toxic pollutant entries into the storm drainage system that will be very difficult to detect and correct. Highly irregular dry-weather outfall flow rates or chemical characteristics could indicate industrial or commercial inappropriate entries into the storm drain system.

During the demonstration phase of this research project (Pitt and Lalor publication pending), odors and high turbidity were found to be the most useful physical indicators of severely contaminated outfall flows. High turbidity correlated well with high levels of surfactants and toxicity. Noticeable odors also correlated well with elevated toxicity. Color was not a very useful indicator of gross contamination and elevated toxicity, unless the color exceed 65 HACH color units.

Gross industrial wastewater contamination may be indicated by the presence and nature of floatable material and deposits near the outfall. Table 11 summarizes possible chemical and physical characteristics of non-stormwater discharges which could come from various industries. The properties considered are pH, total dissolved solids, odor, color, turbidity, floatable materials, vegetation, and damage to outfall structure. The descriptions in each of these categories contain the most likely conditions for a non-stormwater discharge coming from a particular industry. It should be noted that outfalls are likely to be affected by several industrial sources simultaneously, especially if draining industrial parks. The initial watershed analysis, discussed previously, which needs to describe the industrial and commercial facilities that are operating in each outfall's watershed, will be of great assistance in identifying which industries may be contributing dry-weather entries into the storm drainage system.

SIMPLE CHECKLIST FOR MAJOR FLOW COMPONENT IDENTIFICATION

Figure 10 is a flow chart describing the analysis strategy to identify the major non-stormwater discharge sources in residential areas. The first indicator is the presence or absence of flow. If no dry-weather flow exists at an outfall, then indications of intermittent flows must be investigated. Specifically, stains, deposits, odors, unusual stream-side vegetation conditions, and damage to outfall structures can all indicate intermittent non-stormwater flows. However, frequent visits to outfalls over long time periods are needed to confirm that only stormwater flows occur. The other points on the flow chart (Figure 10) serve to indicate if major contaminating sources are present, or if the water is uncontaminated water. The other methods discussed later are needed to quantify the component contributions.

Treated Potable Water

A number of tracer parameters may be useful for distinguishing treated potable water from natural waters:

- Major ions or other chemical/physical characteristics of the flow components can vary substantially depending upon whether the water supply sources are groundwater or surface water, and whether the sources are treated or not. Specific conductance may also serve as a rough indicator of the major water source.

- Fluoride can often be used to separate treated potable water from untreated water sources. Untreated water sources can include local springs, groundwater, regional surface flows or non-potable industrial waters. If the treated water has no fluoride added, or if the natural water has fluoride concentrations close to potable water fluoride concentrations, then fluoride may

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TABLE 11. CHEMICAL AND PHYSICAL PROPERTIES OF INDUSTRIAL NON-STORMWATER ENTRIES INTO STORM DRAINAGE SYSTEMS

Industrial Categories Major Classifications SIC Group Numbers		Odor	Color	Turbidity	Floatables	Debris & Stains	Damage to Outfall Structures	Vegetation	pH	Total Dissolved Solids
Primary Industries										
20	*Food and Kindred Products*									
201	Meat Products	Spoiled Meats Rotten Eggs and Flesh	Brown to Reddish Brown	High	Animal Fats, Byproducts Pieces of Processed Meats	Brown to Black	High	Flourish	Normal	High
202	Dairy Products	Spoiled Milk Rancid Butter	Gray to White	High	Animal Fats Spoiled Milk Products	Gray to Light Brown	High	Flourish	Acidic	High
203	Canned & Preserved Fruits & Vegetables	Decaying Products Compost Pile	Various	High	Vegetable Waxes, Seeds, Skins, Cores, Leaves	Brown	Low	Normal	Wide Range	High
204	Grain Mill Products	Slightly Sweet & Musty Grainy	Brown to Reddish Brown	High	Grain Hulls and Skins Straw & Plant Fragments	Light Brown	Low	Normal	Normal	High
205	Bakery Products	Sweet and or Spoiled	Brown to Black	High	Cooking Oils, Lard, Flour, Sugar	Gray to Light Brown	Low	Normal	Normal	High
206	Sugar and Confectionery Products	NA	NA	Low	Low Potential	White Crystals	Low	Normal	Normal	High
207	Fats and Oils	Spoiled Meats, Lard or Grease	Brown to Black	High	Animal Fats, Lard	Gray to Light Brown	Low	Normal	Normal	High
208	Beverages	Flat Sods, Beer or Wine, Alcohol, Yeast	Various	Moderate	Grains & Hops, Broken Glass, Discarded Canning Items	Light Brown	High	Inhibited	Wide Range	High
21	Tobacco Manufactures	Dried Tobacco, Cigars, Cigarettes	Brown to Black	Low	Tobacco Stems & Leaves Papers and Fillers	Brown	Low	Normal	Normal	Low
22	Textile Mill Products	Wet Burlap, Bleach, Soap, Detergents	Various	High	Fibers, Oils, Grease	Gray to Black	Low	Inhibited	Basic	High
23	Apparel and Other Finished Products	NA	Various	Low	Some Fabric Particles	NA	Low	Normal	Normal	Low
Material Manufacture										
24	Lumber & Wood Products	NA	NA	Low	Some Sawdust	Light Brown	Low	Normal	Normal	Low
25	Furniture & Fixtures	Various	Various	Low	Some Sawdust, Solvents	Light Brown	Low	Normal	Normal	Low
26	Paper & Allied Products	Bleach, Various Chemicals	Various	Moderate	Sawdust, Pulp Paper Waxes, Oils	Light Brown	Low	Normal	Wide Range	Low
27	Printing, Publishing, and Allied Industries	Ink, Solvents	Brown to Black	Moderate	Paper Dust, Solvents	Gray to Light Brown	Low	Inhibited	Normal	High
31	Leather & Leather Products	Leather, Bleach Rotten Eggs or Flesh	Various	High	Animal Flesh & Hair Oils & Grease	Gray to Black Salt Crystals	High	Highly Inhibited	Wide Range	High
33	Primary Metal Industries	Various	Brown to Black	Moderate	Ore, Coke, Limestone Millscale, Oils	Gray to Black	High	Inhibited	Acidic	High
34	Fabricated Metal Products	Detergents, Rotten Eggs	Brown to Black	High	Dirt, Grease, Oils Sand, Clay Dust	Gray to Black	Low	Inhibited	Wide Range	High
32	Stone, Clay, Glass, and Concrete Products	Wet Clay, Mud Detergents	Brown to Reddish-Brown	Moderate	Glass Particles Dust from Clay or Stone	Gray to Light Brown	Low	Normal	Basic	Low

(continued)

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TABLE 11. (continued)

Industrial Categories Major Classifications SIC Group Numbers		Odor	Color	Turbidity	Floatables	Debris & Stains	Damage to Outfall Structures	Vegetation	pH	Total Dissolved Solids
Chemical Manufacture										
28	*Chemicals & Allied Products*									
281 2	Alkalis and Chlorine	Strong Halogen or Chlorine Pungent, Burning	Alkalis - NA Chlorine - Yellow to Green	Moderate	Glass Particles Dust from Clay or Stone	Gray to Light Brown	Highly Inhibited	Normal	Basic	Low
281 6	Inorganic Pigments	NA	Various	High	Low Potential	Various	Low	Highly Inhibited	Wide Range	High
282	Plastic Materials and Synthetics	Pungent, Fishy	Various	High	Plastic Fragments, Pieces of Synthetic Products	Various	Low	Inhibited	Wide Range	High
283	Drugs	NA	Various	High	Gelatin Byproducts for Capsulating Drugs	Various	Low	Highly Inhibited	Normal	High
284	Soap, Detergents, & Cleaning Preparations	Sweet or Flowery	Various	High	Oils, Grease	Gray to Black	Low	Inhibited	Basic	High
285	Paints, Varnishes, Lacquers, Enamels and Allied Products (SB-Solvent Base)	Latex- Ammonia SB-Dependent upon Solvent (Paint Thinner, Mineral Spirits)	Various	High	Latex - NA SB-All Solvents	Gray to Black	Low	Inhibited	Latex- Basic SB- Normal	High
286	*Industrial Organic Chemicals*									
286 1	Gum and Wood Chemicals	Pine Spirits	Brown to Black	High	Rosins and Pine Tars	Gray to Black	Low	Inhibited	Acidic	High
286 5	Cyclic Crudes, & Cyclic Intermediates, Dyes, & Organic Pigments	Sweet Organic Smell	NA	Low	Translucent Sheen	NA	Low	Highly Inhibited	Normal	Low
287	*Agricultural Chemicals*									
287 3	Nitrogenous Fertilizers	NA	NA	Low	NA	White Crystalline Powder	High	Inhibited	Acidic	High
287 4	Phosphatic Fertilizers	Pungent Sweet	Milky White	High	NA	White Emorphous Powder	High	Inhibited	Acidic	High
287 5	Fertilizers, Mixing Only	Various	Brown to Black	High	Pelletized Fertilizers	Brown Emorphous Powder	Low	Normal	Normal	High
29	*Petroleum Refining and Related Industries*									
291	Petroleum Refining	Rotten Eggs Kerosene, Gasoline	Brown to Black	High	Any Crude or Processed Fuel	Black Salt Crystals	Low	Inhibited	Wide Range	High
30	Rubber & Miscellaneous Plastic Products	Rotten Eggs Chlorine, Peroxide	Brown to Black	Moderate	Shredded Rubber Pieces of Fabric or Metal	Gray to Black	Low	Inhibited	Wide Range	High

(continued)

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TABLE 11. (continued)

Industrial Categories Major Classifications SIC Group Numbers	Odor	Color	Turbidity	Floatables	Debris & Stains	Damage to Outfall Structures	Vegetation	pH	Total Dissolved Solids
Transportation & Construction									
15 Building Construction	Various	Brown to Black	High	Oils, Grease, Fuels	Gray to Black	Low	Normal	Normal	High
16 Heavy Construction	Various	Brown to Black	High	Oils, Grease, Fuels Diluted Asphalt or Cement	Gray to Black	Low	Normal	Normal	High
Retail									
52 Building Materials, Hardware, Garden Supply, and Mobile Home Dealers	NA	Brown to Black	Low	Some Seeds, Plant Parts, Dirt, Sawdust, or Oil	Light Brown	Low	Normal	Normal	Low
53 Gen. Merchandise Stores	NA	NA	NA	NA	NA	Low	Normal	Normal	Low
54 Food Stores	Spoiled Produce Rancid, Sour	Various	Low	Fragments of Food Decaying Produce	Light Brown	Low	Flourish	Normal	Low
55 Automotive Dealers & Gasoline Service Stations	Oil or Gasoline	Brown to Black	Moderate	Oil or Gasoline	Brown	Low	Inhibited	Normal	Low
56 Apparel & Accessory Stores	NA	NA	Low	NA	NA	Low	Normal	Normal	Low
57 Home Furniture, Furnishings, & Equipment Stores	NA	NA	Low	NA	NA	Low	Normal	Normal	Low
58 Eating & Drinking Places	Spoiled Foods Oil & Grease	Brown to Black	Low	Spoiled or Leftover Foods	Brown	Low	Normal	Normal	Low
Coal Steam Electric Power	NA	Brown to Black	High	Coal Dust	Black Emorphous Powder	Low	Normal	Slightly Acidic	Low
Nuclear Steam Electric Power	NA	Light Brown	Low	Oils, Lubricants	Light Brown	Low	Normal	Normal	Low

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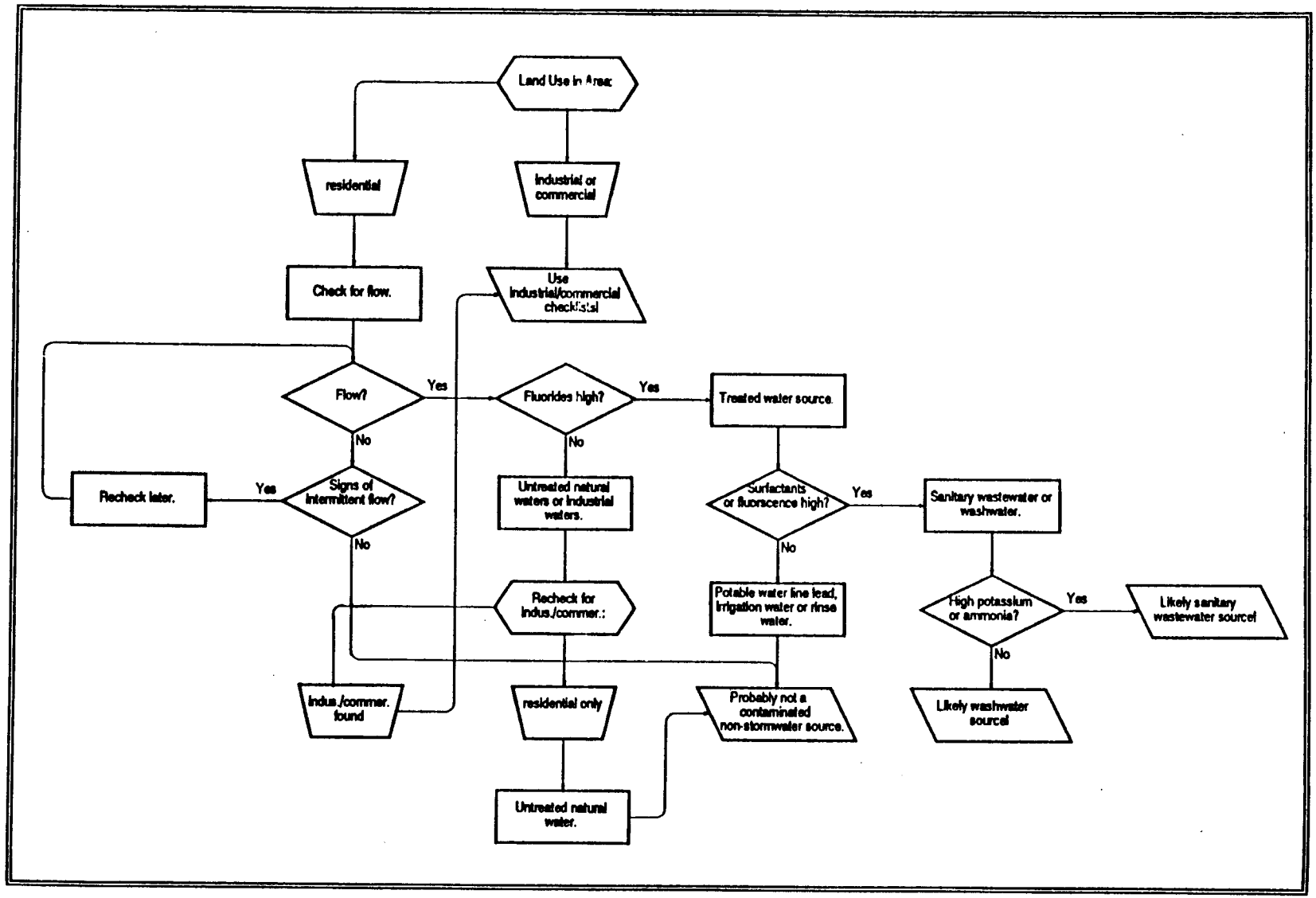

Figure 10. Flow chart to identify residential area non-stormwater flow sources.

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not be an appropriate indicator.

- Hardness can also be used as an indicator if the potable water source and the baseflow are from different water sources. An example would be if the baseflow is from hard groundwater, and the potable water is from softer surface supplies.

- If the concentration of chlorine is high, then a major leak of disinfected potable water is likely to be close to the outfall. Because of the rapid dissipation of chlorine in water (especially if some organic contamination is present) it is not a good parameter for quantifying the amount of treated potable water observed at the outfall.

Water from potable water supplies (that test positive for fluorides, or other suitable tracers) can be relatively uncontaminated, e.g., potable waterline leakage or irrigation runoff, or heavily contaminated, e.g., sanitary wastewater.

Sanitary Wastewaters

In areas containing no industrial or commercial sources, sanitary wastewater is probably the most severe dry-weather contaminating source of storm drain flows. The following parameters can be used for quantifying the sanitary wastewater components of the treated potable water portion:

- Surfactant analyses may be useful in determining the presence of sanitary wastewaters. However, surfactants present in water originating from potable water sources could indicate sanitary wastewaters, laundry wastewaters, car washing wastewater, or any other waters containing surfactants. If surfactants (or fluorescence) are not present, then the potable water could be relatively uncontaminated (potable waterline leaks or irrigation runoff).

- The presence of fabric whiteners (as measured by fluorescence using a fluorometer in the laboratory or in the field) can also be used in distinguishing laundry and sanitary wastewaters.

- Sanitary wastewaters often exhibit predictable trends during the day in flow and quality. In order to maximize the ability to detect direct sanitary wastewater connections into the storm drainage system, it would be best to survey the outfalls during periods of highest sanitary wastewater flows (mid to late morning hours).

- The ratio of surfactants to ammonia or potassium concentrations may be an effective indicator of the presence of sanitary wastewaters or septic tank effluents. If the surfactant concentrations are high, but the ammonia and potassium concentrations are low, then the contaminated source may be laundry wastewaters. Conversely, if ammonia, potassium, and surfactant concentrations are all high, then sanitary wastewater is the likely source. Some researchers have reported low surfactants in septic tank effluents. Therefore, if surfactants are low, but potassium and ammonia are both high, septic tank effluent may be present. However, Pitt and Lalor (publication pending) found high surfactant concentrations in septic tank effluent during the Birmingham, Alabama demonstration project. This further stresses the need to obtain local site specific characterization data for potential contaminating sources.

- Obviously, odor and other physical characteristics, e.g., turbidity, coarse and floating "tell-tale" solids, foaming, color, and temperature would also be very useful in distinguishing sanitary wastewater from washwater or laundry wastewater sources. However, these indicators may not be very obvious for small levels of sanitary wastewater contamination.

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FLOW-WEIGHTED MIXING CALCULATIONS

Before any flow-weighted mixing calculations can be made, the characteristics of potential contaminating sources must be identified. Table 12 summarizes hypothetical concentration medians and COVs for tracers that have been recommended to be used in the investigation of non-stormwater entries into storm drainage systems in residential areas. This method is an extension of the checklistmethod described above and attempts to quantify the likely source flow components at the outfall during dry weather.

Two general groupings of flow sources can usually be recognized for each of these tracers, a high concentration group and a low concentration group. Table 13 describes these groups, along with their composite tracer concentration ranges, variations, and medians. The outfall flow can be split between the two general groupings by simple algebra. This method can result in substantial errors if the tracer concentrations cannot be separated into distinct source groupings. The next two methods, using matrix algebra to solve simultaneous equations, do not require this simplifying assumption.

Example Calculations

The drainage area for a sampled outfall had no septic tanks or commercial and industrial land uses. The likely flow sources had source flow characteristics as described in Table 12. The required detection limits and precision for outfall characterizations must be determined, as previously described, for these source flow characteristics and desired study results. This outfall had the following tracer concentrations in a dry-weather sample:

Fluoride: 0.6 mg/L

Hardness: 200 mg/L as $CaCO_3$

Surfactants: 0.6 mg/L as MBAS

Potassium: 3 mg/L

Ammonia: 3 mg/L

The water had a slight septic odor, with some floatables of apparent sanitary wastewater origin. In addition, dry-weather flow was observed at the outfall during all visits.

It is apparent that this outfall has a direct connection(s) of raw sanitary wastewater. This method can determine the approximate mix of sanitary wastewater in the outfall flow and identify the other flow components. Table 14 summarizes the example calculations used in this analysis. The list below indicates the approximate expected source components at this outfall from this analysis:

Raw sanitary wastewater: 5%

Laundry wastewater: 5%

Groundwater: 70%

Remainder (most likely potable water, but may also contain irrigation water): 20%

This analysis did not consider the potential ranges in observed tracer concentrations and the

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TABLE 12. ASSUMED SOURCE FLOW QUALITY
(All Conc. in mg/L)

Source		Fluoride	Hardness (as Ca Co_3)	Surfactants (as MBAS)	Potassium	Ammonia (N as NH_3)
Surface Waters	median	0.14	39	0.35	0.72	0.76
	COV	0.23	0.20	0.13	0.23	1.1
Ground-waters	median	0.29	250	0.05	1.7	0.22
	COV	0.23	0.14	0.13	0.40	0.63
Septic Tank Effluent	median	1.3	39	0.05	21	47
	COV	0.14	0.20	0.13	0.91	1.5
Raw Sanitary Wastewater	median	1.3	39	4.6	21	22
	COV	0.14	0.20	2.2	0.91	0.63
Laundry Wastewater	median	1.3	39	4.6	5.3	0.31
	COV	0.14	0.20	2.2	0.57	0.91
Irrigation Water	median	1.3	39	0.35	0.72	0.38
	COV	0.14	0.20	0.13	0.23	1.1

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TABLE 13. CHARACTERISTICS OF SOURCE GROUPINGS

Fluorides		
surface & groundwaters		all other categories
overall range:	0.1→0.4 mg/L	1→1.5 mg/L
COV:	0.54	0.14
median:	0.20 mg/L	1.3 mg/L
Concentration ratio of medians:	6.5	
Hardness		
groundwaters		all other categories
overall range:	200→300 mg/L	30→50 mg/L
COV:	0.14	0.20
median:	250 mg/L	39 mg/L
Concentration ratio of medians:	6.4	
Surfactants		
raw sanitary wastewater & laundry wastewater		all other categories
overall range:	0.2→100 mg/L	0.04→0.4 mg/L
COV:	2.2	0.83
median:	4.6 mg/L	0.14 mg/L
Concentration ratio of medians:	33	
Potassium		
septic tank effluent & raw sanitary wastewater		all other categories
overall range:	10→100 mg/L	0.5→11 mg/L
COV:	0.91	1.2
median:	21 mg/L	2.3 mg/L
Concentration ratio of medians:	9.1	
Ammonia		
septic tank effluent & raw sanitary wastewater		all other categories
overall range:	6→380 mg/L	0.1→3 mg/L
COV:	1.5	1.3
median:	47 mg/L	0.44 mg/L
Concentration ratio of medians:	107	

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TABLE 14. MIXTURE CALCULATIONS TO IDENTIFY SOURCE FLOW COMPONENTS

Fluorides: 0.6 mg/L observed at outfall

x = fraction of surface & groundwater
with concentration of 0.2 mg/L
y = fraction of treated water (all other sources)
with concentration of 1.3 mg/L
(x & y fraction concentrations taken from Table 13)

$x(0.2) + y(1.3) = 0.6$ (for a unit volume of outfall water)
$x + y = 1$ (for no other sources of fluorides)

x = 0.63 (surface & groundwater)
y = 0.37 (all other sources)

Hardness 200 mg/L as $CaCO_3$ observed at outfall

x = fraction of groundwater
with concentration of 250 mg/L as $CaCO_3$
y = fraction of all other sources
with concentration of 39 mg/L as $CaCO_3$

$x(250) + y(39) = 200$

x = 0.76 (groundwater)
y = 0.24 (all other sources)

<u>From Fluorides and Hardness Data:</u>

Groundwater & Surface water =	0.63
Groundwater alone =	0.76
Surface water alone =	-0.13→0

Therefore:
Groundwater fraction = (0.63 + 0.76)/2 = 0.7

Surfactants: 0.6 mg/L as MBAS observed at outfall

x = fraction of sanitary & laundry wastewater
with a concentration of 4.6 mg/L as MBAS
y = fraction of all other sources
with a concentration of 0.14 mg/L as MBAS

$x(4.6) + y(0.14) = 0.6$

x = 0.10 (sanitary & laundry wastewater)
y = 0.90 (all other sources)

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TABLE 14. (continued)

Potassium: 3 mg/L observed at outfall

x = fraction of sanitary wastewater
with a concentration of 21 mg/L
y = fraction of all other sources
with a concentration of 2.3 mg/L

$$x(21) + y(2.3) = 3$$

x = 0.04 (sanitary wastewater)
y = 0.96 (all other sources)

Ammonia: 3 mg/L observed at outfall

x = fraction of sanitary wastewater
with a concentration of 47 mg/L
y = fraction of all other sources
with a concentration of 0.44 mg/L

$$x(47) + y(0.44) = 3$$

x = 0.06 (sanitary wastewater)
y = 0.94 (all other sources)

From Surfactants, Potassium, and Ammonia Data:

Sanitary wastewater = (0.04 + 0.06)/2 = 0.05

Laundry wastewater = 0.1 - 0.05 = 0.05

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resulting errors that may be associated with the above mixture portions. The following procedures are better suited for error analyses.

MATRIX ALGEBRA SOLUTION OF SIMULTANEOUS EQUATIONS

It is possible to estimate the outfall source flow components using a set of simultaneous equations. The number of unknowns should equal the number of equations available, resulting in a square matrix. If there are eleven likely source categories, then there should be eleven tracer parameters used. If there are only four possible sources, then only four tracer parameters should be used.

Further statistical analyses may therefore be needed to rank the usefulness of the tracers for distinguishing different flow sources. Pitt and Lalor (publication pending) show examples of how cluster and principal component analyses can be used to identify redundancy and other problems in the data library. As an example, chlorine is not useful for these analyses because the concentration variability within many source categories is high (it is also not a conservative parameter). Chlorine may still be a useful parameter, but only to identify possible large potable waterline leaks. It cannot be used to quantify the flow components. Another parameter having problems for most situations is pH. The variation of pH between sources is very low (they are all very similar). However, pH may still be useful to identify industrial wastewater problems, but it cannot be used to quantify flow components. pH is also not linearly affected by mass balance mixtures (a solution of 50 percent/50 percent of two components would not result in a pH value that is the average of the two individual pH values).

These equations are structured on a mass balance basis, like the previous procedure, but they can be used to distinguish all source categories simultaneously. A simplified example is shown in the following discussion considering just four possible flow components and four tracer parameters (P1, P2, P3, P4). This would result in the following set of equations for each outfall sample:

	possible sources:									
tracer parameter:	1		2		3		4			outfall quality
P1:	(A1)(C11)	+	(A2)(C21)	+	(A3)(C31)	+	(A4)(C41)	=		m1
P2:	(A1)(C12)	+	(A2)(C22)	+	(A3)(C32)	+	(A4)(C42)	=		m2
P3:	(A1)(C13)	+	(A2)(C23)	+	(A3)(C33)	+	(A4)(C43)	=		m3
P4:	(A1)(C14)	+	(A2)(C24)	+	(A3)(C34)	+	(A4)(C44)	=		m4

A1 through A4 represent the fraction of flow contributed from each possible flow source. The "C" terms represent concentrations from the source flow library for each particular parameter (P) within each flow source(1-4). The "m" terms represent the concentration of P actually measured in the outfall sample.

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The following is an example for an outfall dry-weather sample:

tracer parameter:	potable water	ground water	sanitary wastewater	laundry wastewater	outfall quality
	possible sources:				
fluoride:	(A1)(0.97 mg/L) +	(A2)(0.031 mg/L) +	(A3)(0.77 mg/L) +	(A4)(33 mg/L)	= 3.8 mg/L
hardness:	(A1)(49 mg/L) +	(A2)(240 mg/L) +	(A3)(140 mg/L) +	(A4)(14 mg/L)	= 126 mg/L
surfactants:	(A1)(0 mg/L) +	(A2)(0 mg/L) +	(A3)(1.5 mg/L) +	(A4)(27 mg/L)	= 3.0 mg/L
potassium:	(A1)(1.6 mg/L) +	(A2)(0.73 mg/L) +	(A3)(6.0 mg/L) +	(A4)(3.5 mg/L)	= 2.2 mg/L

This simple 4x4 matrix can be solved using available scientific calculators or math programs for personal computers, or by hand. For this example, the following are the approximate flow components (rounded to the nearest 5 percent):

- treated potable water (A1): 30%
- groundwater (A2): 35%
- sanitary wastewater (A3): 20%
- laundry wastewater (A4): 10%

These component contributions do not all add up to 100 percent. A number of errors, especially variations in source area characteristics and other sources present that were not considered, tend to result in component sums that are not 100 percent. The following method is similar, but considers uncertainty in source area characteristics and results in a range of likely component contributions.

MATRIX ALGEBRA CONSIDERING PROBABILITY DISTRIBUTIONS OF LIBRARY DATA

A stochastic version of the above procedure enables the variation in the library values to be considered. The matrix is set up in the same way, but instead of using a single value representing the parameter concentration for each likely source flow, a Monte Carlo simulation is used to randomly select values. A large number of analyses (from a few hundred to many thousands) are conducted and the percentage contributions for each component source are presented as a probability distribution instead of a single value.

It is therefore necessary to describe the distribution of source flow characteristics. In most cases, the tracer parameters can be represented using log-normal distributions. Some parameters, however, are adequately described with normal distributions. Again, local source flow monitoring is necessary to obtain this information. Pitt and Lalor (publication pending) contains examples using this method, including the code for the necessary computer program.

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SECTION 7

WATERSHED SURVEYS TO CONFIRM AND LOCATE INAPPROPRIATE POLLUTANT ENTRIES TO THE STORM DRAINAGE SYSTEM

After initial outfall surveys have indicated the presence of contamination, further detailed analyses are needed to identify and locate the specific contaminant source(s) (e.g., residential, commercial, and/or industrial) in the drainage area. For source identification and location, upstream survey techniques should be used in conjunction with an in-depth watershed evaluation. Information on watershed activities can be obtained from aerial photography and/or zoning maps, while upstream survey techniques will include the analysis of the dry-weather flow at several manhole points along the storm drainage system to narrow the location of the contaminating source; tests for specific pollutants or ions associated with known activities within the outfall catchment area; and the measurement of water flow rate and temperature, visual and T.V. inspections, and smoke and dye tests.

USING TRACER PARAMETERS IN THE DRAINAGE SYSTEM

In order to identify the specific contaminant sources in the drainage system, further detailed watershed analyses are needed. These may include:

- drainage system surveys (tests for specific pollutants, visual inspections, T.V. drainage pipe inspections, and smoke and dye tests),
- in-depth watershed evaluation (including aerial photographs), and
- industrial and commercial site studies.

Review Industrial User Surveys or Reports

This will require the submission of a questionnaire to industries to determine which industries or commercial locations are discharging to a storm drainage system. However site inspections will still be required because questionnaires may not be returned or may give incorrect details (either deliberately or unknowingly).

Follow-up Drainage Area and On-Site Investigations.

Further drainage area investigations upstream of identified problem outfalls would be conducted after the outfall studies have indicated dry-weather discharge problems. In order to be cost-effective, only a sub-sample of manholes located in a drainage area identified as having significant non-stormwater sources should be tested for the tracers. As an example, the main storm drain trunk sewer could be divided into tenths and the manholes closest to these subdivisions would be sampled. This would identify the upper limit of the drainage area above which the major sources are not located. A location may also be identified where the downstream manhole tracer mass yields (concentration times flow rate) are the same. This would mark the downstream limit of the contributing area for the tracers of concern. After the main trunk drainage reach is identified that contains the major non-stormwater sources, the branch storm drain lines can be similarly subdivided (but into fewer sections each, perhaps about three) and evaluated. Depending on the drainage area and complexity

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of the storm drainage system, this scheme could be suitably modified to enable the identification of relatively small areas responsible for the non-stormwater pollutant entries into the storm drainage system. These small areas would then be subject to the more intensive on-site investigations by smoke tests, dye studies, and T.V. inspections.

The above drainage system analysis procedure may find that the drainage system is contaminated by widespread sanitary wastewater entries, possibly due to sanitary and storm drainage systems in extremely poor condition. This situation may require that the drainage system undergo extensive and costly repairs. It may be more appropriate to consider the storm drainage system as a combined sewer and examine control alternatives that have been developed for combined sewer systems. This would also save further detailed drainage system analyses costs.

These drainage system surveys would be followed by industrial and commercial on-site investigations (e.g., dye and smoke studies and T.V. inspections) to locate specific sources of non-stormwater pollutant entries into the drainage system. Additionally, aerial photography can be very useful during later phases of non-stormwater discharge control projects. As an example, aerial photography can help identify areas having failing septic systems located in residential areas served by storm drainage systems. Aerial photography can also be used to identify continuous discharges to surface drainage systems, such as sump discharges, and to identify storage areas that may be contributing significant amounts of pollutants during rains. For example, the Tennessee Valley Authority (TVA), among other agencies, has extensively used aerial photography (stereo color infrared) to identify pollution sources, especially from failing septic tanks (Perchalski and Higgins 1988). The TVA's flights are made in early spring when investigating septic tank failures, to be able to identify unusual grass conditions, with minimal interference from trees. The flights are made at 6,000 feet, with resulting image scales of 1 inch to 1,000 feet. Their photography costs have been about $40 to $150 per square mile.

FLOW MASS BALANCES, DYE STUDIES, AND SMOKE TESTS

Industrial areas are known to contribute significantly polluted wet-weather stormwater discharges, along with contaminated dry-weather entries into the storm drainage system. Additional industrial site investigations are therefore needed to identify activities that apparently contribute these contaminants to the storm drainage system. Figure 11 is an industrial site survey form prepared by the Non-Point Source and Land Management Section of the Wisconsin Department of Natural Resources (R. Bannerman, personal communication). This form has been used to help identify industrial activities that contribute significantly polluted, indirectly connected dry- and wet-weather non-stormwater entries into the storm drainage system.

This form only considers outside sources that would affect the storm drainage system by entering through inlets or through sheetflow runoff into drainage channels. It does not include any information concerning indoor activities, or direct plumbing connections to the storm drainage system. However, the information included on this form can be very helpful in devising runoff control programs for industrial areas. This information most likely affects wet-weather discharges much more than dry-weather discharges. Obvious dry-weather leaching or spillage problems are also noted on the form.

Locating An Industrial Source

Hypothetical examples have been created to demonstrate how dry-weather discharges can be characterized so that their likely industrial sources can be identified. These examples show how observations of outfall conditions and simple chemical analyses, combined with a basic knowledge of wastewater characteristics of industrial and commercial operations located in the drainage area, can

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City:_____ Industry Name:_____
Site Number:_____ Photo #_____
Street Address:_____ Roll #_____
Type of industry:_____
Instructions: Fill in blanks or circle best answer in following:

Material/waste Storage Areas
1. Type of material/waste: _____
2. Method of storage: pile tank dumpster other _____
3. Area occupied by material/waste (acres): _____
4. Type of surface under material/waste: paved unpaved
5. Material/waste is disturbed: often sometimes never unsure
6. Description of spills (material, quantity & frequency): _____
7. Nearest drainage (feet) and drainage type: _____
8. Control practice: berm tarp buffer none other _____
9. Tributary drainage area, including roofs (acres) _____
10. Does storage area drain to parking lot: yes no unsure

Heavy equipment storage
1. Type of equipment:_____
2. Area covered by equipment (acres): _____
3. Type of surface under equipment: paved unpaved
4. Nearest drainage (feet) and drainage type: _____
5. Control practice: berm tarp buffer none other _____
6. Tributary drainage area, including roofs (acres) _____
7. Does storage area drain to parking lot: yes no unsure

Air pollution
1. Description of setteable air pollutants (types & quantities): _____
2. Description of particulate air pollutant controls: _____

Railroad yard
1. Size of yard (number of tracks): _____
2. General condition of yard: _____
3. Description of spills in yard (material, quantity & frequency): _____
4. Type of surface in yard: paved unpaved
5. Nearest drainage (feet) and drainage type: _____
6. Type of control practice: berm buffer other _____
7. Does yard drain to parking lot: yes no unsure
8. Tributary drainage area, including roofs (acres): _____

Loading Docks
1. Number of truck bays: _____
2. Type of surface: paved unpaved
3. Description of spills in yard (material, quantity & frequency): _____
4. Nearest drainage (feet) and drainage type: _____
5. Type of control practice: berm buffer other _____
6. Does loading area drain to parking lot: yes no unsure
7. Tributary drainage area, including roofs (acres): _____

Source: From Wisconsin Dept. of Natural Resources (R. Bannerman, Personal communication)

Figure 11. Industrial Inventory Field Sheet. (Use other sheets for multiple areas on same site)

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be used to identify the possible pollutant sources. The initial activities include pollutant analyses of outfalls being investigated. This requires the characterization of the non-stormwater flows, the identification of the likely industries responsible for the observed discharges, and finally, locating the possible specific sources in the watershed.

Hypothetical Conditions--

The hypothetical industries which were identified as being located in a stormwater drainage area (from the watershed analysis) included a vegetable cannery, general food store, fast food restaurant, cheese factory, used car dealer, cardboard box producer, and a wood treatment company. The methods used to determine the most likely industrial source of the dry-weather discharges are considered for three hypothetical situations of outfall contamination.

Case Example One--The hypothetical results of the pollutant analysis for the first situation found constant dry-weather flow at the outfall. The measurements indicated a normal pH (6) and low total dissolved solids concentrations (300 mg/L). Other outfall characteristics included a strong odor of bleach, no distinguishing color, moderate turbidity, sawdust floatables, a small amount of structural corrosion, and normal vegetation.

The significant characteristic in this situation is the sawdust floatables (see Figure 12). The industries which could produce sawdust and have dry-weather flow drainage to this pipe are the cardboard box company and the wood treatment company. According to SIC code, the cardboard box company would fall under the category of "Paper Products" (SIC# 26) while the wood treatment company would be under that of "Lumber and Wood" products (SIC# 24). Looking up these two industries by their corresponding SIC group numbers in Table 11 and comparing the listed properties, indicates that the paper industry has a strong potential for the odor of bleach. Wood products does not indicate any particular smell.

Based upon this data, the most likely industrial source of the industrial non-stormwater discharge would be the cardboard box company. Table 2 under SIC# 26 indicates that there is a high potential for direct connections in paper industries under the categories of water usage and illicit or inadvertent connections. At this point, further testing should be conducted at the cardboard box company to find if the constant source of contamination is coming from cooling waters, process waters, or direct piping connections (process waters are the most likely source given the bleach and sawdust characteristics).

Case Example 2--The results of the pollutant analysis for the second situation found intermittent dry-weather discharges at the outfall. The test measurements indicated a low pH (3) and high total dissolved solids concentrations (approximately 6,000 mg/L). Other characteristics included a rancid-sour odor, grayish color, high turbidity, gray deposits containing white gelatin-like floatable material, structural damage in the form of spalling concrete, and an unusually large amount of plant life.

The rancid-sour smell and the presence of floatable substances at this outfall indicates that some type of food product is probably spoiling. This narrows the possible suspect industries to the fast food restaurant, cheese factory, vegetable cannery, and food store (see Figure 13). The corresponding SIC categories for each of these industries are "Eating and Drinking Places" (SIC# 58), "Dairy Products" (SIC# 202), "Canned and Preserved Fruits and Vegetables" (SIC# 203), and "Food Stores" (SIC# 54). Comparison of the properties listed in Table 11 for these SIC numbers indicates that elevated plant life is common to industrial wastes for the "Dairy Products" and "Food Stores" categories. However, the deciding factor is the low pH, which is only listed for "Dairy Products". Thus, the white gelatin-like floatables are most likely spoiled cheese byproducts which are also the probable cause of the sour-rancid smell.

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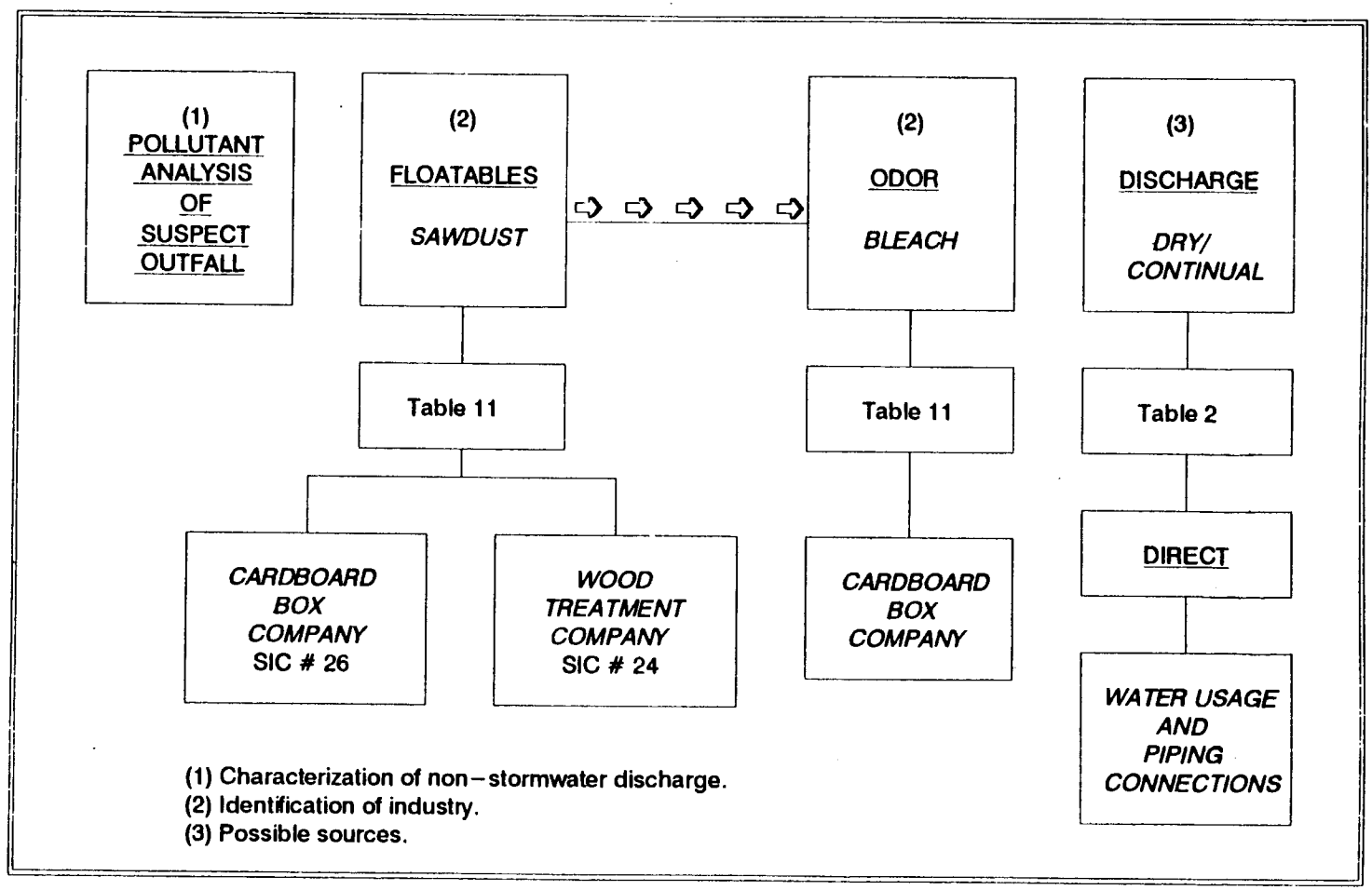

Figure 12. Flowsheet for Industrial Case 1.

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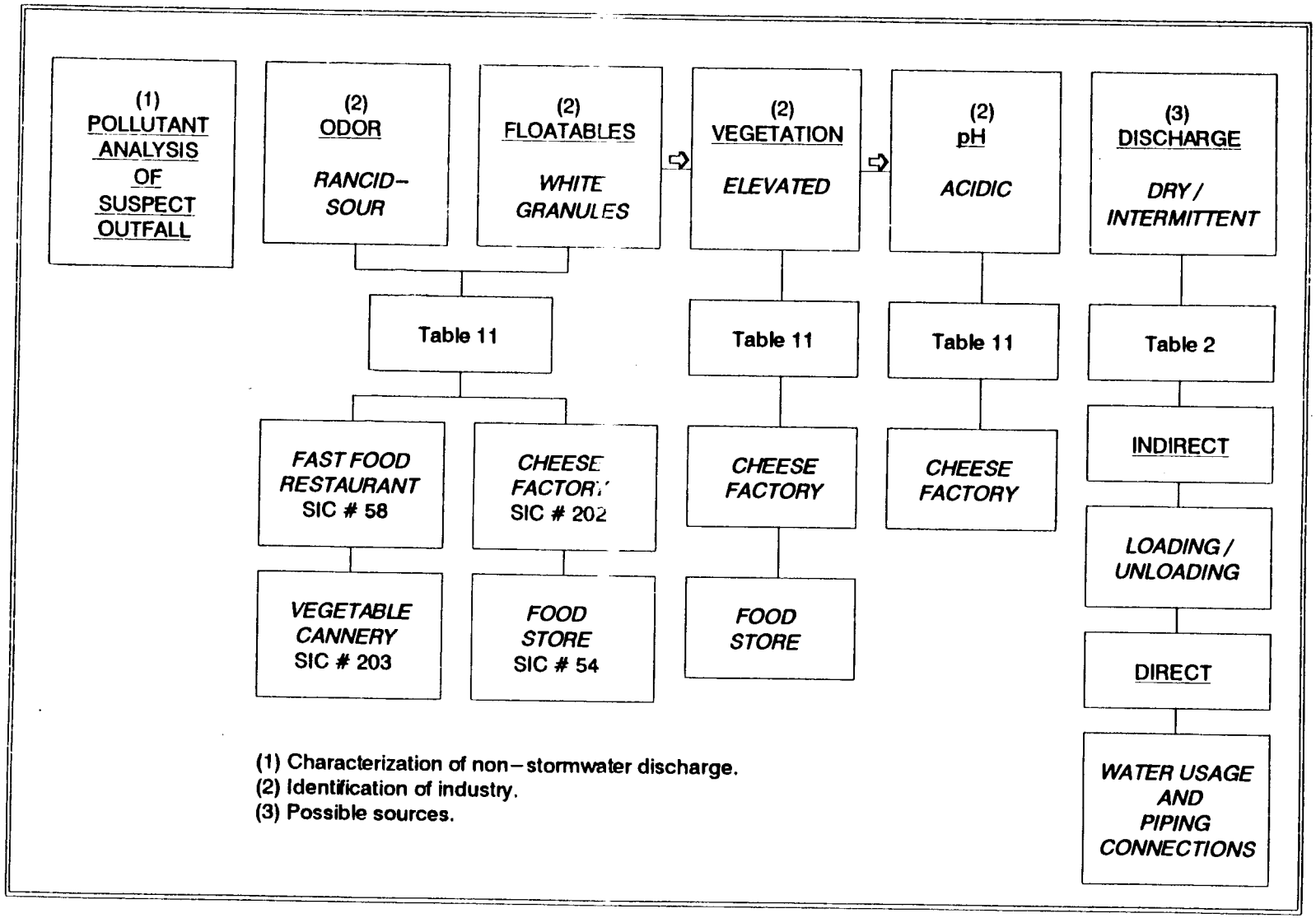

Figure 13. Flowsheet for Industrial Case Example 2.

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Since the dry-weather entry to the storm drainage system occurs intermittently, the flow could be caused by either a direct or indirect connection. To locate the ultimate source of this discharge coming from the cheese factory, both direct and indirect industrial situations are considered under the category of "Dairy Products" in Table 2. Thus, further examination of the loading dock procedures, water usage, and direct piping connections should be conducted since these categories all exhibit high potential for pollution in dairy production.

Case Example 3-- The results of the test measurements for the final situation found a normal pH (6) and low total dissolved solids (about 500 mg/L). Signs of contaminated discharges were found at the outfall only during and immediately following rainfalls. Other outfall properties observed included an odor of oil, deep brown to black color, a floating oil film, no structural damage, and inhibited plant growth (see Figure 14).

According to Table 11, the fast food restaurant and the used car dealer are the only two industrial sources in this area with high potential for causing oily discharges. Their respective SIC categories are "Eating and Drinking Places" (SIC# 58) and "Automotive Dealers" (SIC# 55). Comparison of the properties shown on Table 11 indicates inhibited vegetation only for the second category. Thus, the most likely source of the discharge is the used car dealer.

Furthermore, the source of contamination must likely be indirect, since the discharge occurs only during wet weather. Reference to Table 2, under the category of "Automotive Dealers", indicates a high potential for contamination due to outdoor storage. This fact, plus the knowledge that most used cars are displayed outdoors, makes it fairly clear that surface runoff is probably carrying spilled car oil into the storm drain during rains.

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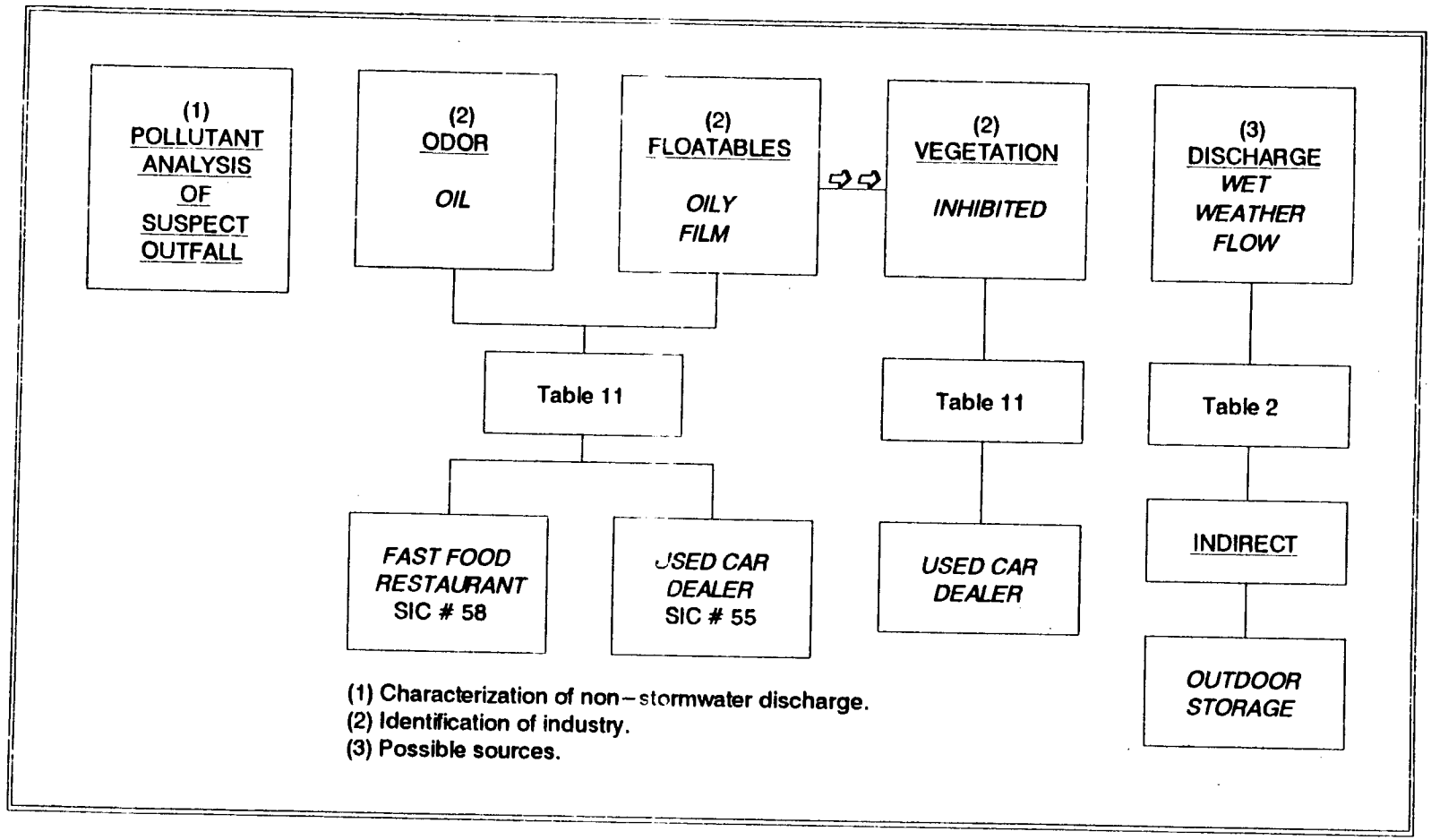

Figure 14. Flowsheet for Industrial Case Example 3.

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SECTION 8

CORRECTIVE TECHNIQUES

In addition to identifying problems of unauthorized or inappropriate entries to stormwater systems, it is even more important to prevent problems from developing at all, and to provide an environment in which future problems will be avoided. Thus, a combined approach of identifying and correcting existing problems and avoiding future problems has considerable merit. In this section, the focus is on discussing ways in which future problems can be avoided. However it should be noted that this is not an in depth review, but has been included to provide the reader with suggestions that could be incorporated into a pollution prevention program.

There are also situations in which the sanitary system is so connected to the stormwater system that good intentions, vigilance, and reasonable remedial actions will not be sufficient to solve the problems. In an extreme case, it may be that while it was thought that a community had a separate sanitary sewer system and a separate storm drainage system, in reality the storm drainage system is acting as a combined sewer system. When recognized for what it really is, the alternatives for the future become clearer: undertake the considerable investment and commitment to rebuild the system as a truly separate system, or recognize the system as a combined sewer system, and operate it as such, without the disillusionment that it is a problem-plagued storm drainage system which can be rehabilitated.

Less extreme than designating a polluted stormwater drainage system a combined sewer system, is the action of focusing on pollution prevention by:

- public education,
- an organized systematic program of disconnecting commercial and industrial non-stormwater entries into the storm drainage system,
- tackling the problem of widespread septic system failure,
- disconnecting direct sanitary sewerage connections,
- rehabilitating storm or sanitary sewers to abate contaminated water infiltration, and
- developing zoning and ordinances.

In this section, the above items will be discussed, together with a section on treatment of wide spread sanitary sewerage failure.

PUBLIC EDUCATION

One can argue that an ill informed and apathetic public has condoned the past actions of private citizens, commercial entities, industrial concerns, and public officials which led to some of the past and present problems with unauthorized entries to storm drainage systems. One also knows the power of an aroused, concerned public in altering behavior at all levels. Thus, public education has a role to play. It can be effective in altering the behavior of an individual who had assumed that the inlet on the curb was the place to discharge used crankcase oil. It can be effective when organized groups lobby for the return of a stream or a reservoir to a clean and attractive condition.

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Public education carries with it the implicit assumption that an educated public will make the "right" decisions, the educated public will be concerned about the "right" problems, and it will encourage private and public organizations to develop solutions to the "right" problems. Fortunately, most of the problems, issues, and corrective measures are clear cut with respect to unauthorized entries to the stormwater system. Public education is a communication art associated with significant changes when successful, and imperceptible change when unsuccessful. As with all education, it does not end, but is a continuing process. The following paragraphs describe some of the ways in which public officials can help to educate the public. The "public" has been subdivided into categories which are representative of the problem areas with respect to unauthorized entries to storm drainage systems. The subcategories of the public are:

- industrial
- commercial
- residential
- governmental

Industrial decision makers can be educated by public officials through direct contact when they seek information, by education of the consultants from whom industry seeks advice, and by education of trade associations. Indirect educational opportunities are provided by speaking to meetings of professional organizations and by writing in professional newsletters and journals. Industrial decision makers are a small group which is likely to respond as they recognize that they have to address the problem of unauthorized entries to the stormwater system.

Commercial storm drainage system users are a larger group to educate. The educational process will have to focus on both proprietors and their employees. It will have to recognize the state of both groups, new businesses opening; existing businesses moving, expanding, and closing; and employees entering the work force and changing jobs. Education will have to be focused in the local community. The role of trade and professional associations will be less than was the case with industrial groups. News announcements in the local press will play a role as well as mailed news items. Individual contact between a public official and the proprietor of a commercial establishment will play a larger role. Follow up and repeated contact may be necessary to answer questions and cope with employee turnover. Public education can also benefit from failures. For example, certain violations of discharge practices may be so serious, or flagrant, that a citation or fine results. The local press, if informed, may find such an incident newsworthy. The general public, or other potential offenders, may benefit from this educational procedure.

An informed public willing to act on their convictions is the product sought from public education. The public educator focuses on large groups, as one-on-one contact is unlikely to be either time or cost effective. Long range educational goals may be tackled through school programs, while shorter range educational goals may focus on community groups. Public education will have to focus on broader environmental issues than inappropriate entries to storm drains. Subgroups in the community may play important roles in public education. For example, scouts may undertake community improvement projects including placing signs on curbside storm drains informing the public that the drain is for stormwater only, and not for discharge of wastes. Thus, public education must take advantage of opportunities presented by groups looking for community improvement projects, the opportunities that are available in working with the school system, and opportunities arising from the news media being supplied with newsworthy items.

The final group that public officials should address in public education is other public officials and governmental institutions. Some small governmental units may not know about precautions to be taken with discharges to storm drainage systems unless they are properly informed. Such subgroups may include road departments, sanitation workers, and workers at public institutions such as hospitals and

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prisons. A multilevel, multitarget public education program can help to avoid problems.

COMMERCIAL AND INDUSTRIAL SITE DISCONNECTIONS OF NON-STORMWATER SOURCES

Out of convenience and out of ignorance, commercial and industrial sites may impose an increasing load on the storm drainage system. This may be through direct discharges to the storm drainage system, or it may be through diffuse and indirect sources in which the site grounds are contaminated by spills and discharges which are then washed off by storm runoff to the storm drain during rainfall events or by washwater during wash-down operations. The problem is compounded by the vast array of sizes of commercial and industrial enterprises. A single person enterprise has little opportunity to build expertise on the subject of stormwater pollution, while a large industrial enterprise may have an environmental division. To the uninformed person, any curb opening may be thought to be part of a comprehensive sanitary wastewater treatment system and the proper entrance point for polluted water discharges or other debris.

Corrective measures for improper uses of storm drains have to be developed recognizing the differences in knowledge and sophistication of the client. Industrial users are relatively few in number but are expected to have the most complex problems. If industrial users are aware, or made aware, of existing and or new federal, state, or local regulations to prevent pollution of stormwater drainage systems, they will usually comply with the regulation. If not, these regulations provide the authority and communication means to instigate corrective action.

Commercial groups are heterogeneous. An appropriate way of working with them to institute changes in their use of storm drainage systems, may be to work with one category of commercial groups at a time. For example, consider gasoline filling stations as a single category. It is possible to focus on correcting similar problems at many facilities that exist in this category. The flushing of radiators may be seasonally common. A typical practice is to let radiator flushing waters (including coolants) to drain to an inlet to the storm drainage system. Education followed by assurance that there will be strict enforcement of discharge regulations or ordinances may be effective. However, a group such as gasoline filling stations cannot be expected to have a long institutional memory as new operators take over and others drop out. Thus, vigilance and follow-up are important to insure that there is not a gradual diminution of appropriate practices.

For both small commercial and large industrial enterprises, willful and knowledgeable violation of the regulations limiting entries to storm drainage systems have to be dealt with firmly and promptly or the enforcement program runs the chance of becoming ineffective. Thus the governmental unit undertaking responsibility for improving the practices regarding entries to storm drainage systems must have an enforcement plan ready.

FAILING SEPTIC TANK SYSTEMS

Failing septic tank systems can have an impact on an otherwise well functioning storm drainage system. Before discussing corrective measures, it is important to identify the relationship that may develop between a septic tank system and a storm drainage system.

A septic tank system consists of two major components: a septic tank and a leaching field (a waste spreading or soil absorption system). In addition, of course, there is piping associated with the system. Sanitary wastewaters are piped directly to the septic tank. The septic tank typically is made of concrete, is rectangular in shape, is usually divided into two compartments, and has a capacity of one to several thousand gallons. The septic tank serves as an anaerobic digestion, floatation and

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settling unit in which biological action converts the biodegradable liquid and solid waste particles into stable end products. Gravity separates a significant portion of both biodegradable and non-biodegradable particulate matter to the tank bottom or top (depending on whether the particles sink or rise, respectively). Some of the products of this partial treatment process are carbon dioxide, methane, hydrogen sulfide and other odor producing gases, digested and refractory or relatively non-biodegradable sludge, and floating scum. Because the septic tank remains full, it must discharge a volume of wastewater each time a volume of wastewater is discharged into it. This discharged water enters a leaching field where some additional treatment occurs and the final effluent is discharged to the ground.

A septic tank may be a low maintenance treatment unit, but it is not entirely maintenance free. As the septic tank continues to be loaded, the scum and sludge layers build up so that the remaining volume available for treatment is reduced. Thus, some of the partially digested or undigested solids, scum, and sludge may be carried from the septic tank to the leaching field where the soil void space may become clogged. As the soil voids become clogged, the ability of the leaching field to handle the liquid portion of the waste is reduced, and surface ponding of the wastewater may result. Of course, ponding could have been prevented by having the septic tank serviced; that is, by having the septic tank pumped. Pumping removes the sludge, scum, and other contents of the septic tank so that its storage and treatment capacity is restored. Pumping frequency varies depending on the size of the septic tank and its loading rate. Residential septic tanks may need to be pumped every two to five years. Commercial and institutional septic tanks may need more frequent pumping.

Failed septic tank systems have the potential to pollute stormwater because the leaching field will saturate the ground, and possibly form ponded water on the ground surface. The ponded water may run off and enter a storm drain inlet or drainage ditch, or infiltrate the ground in another area which is intercepted by a storm drain through infiltration. When it rains, any remaining ponded water may be washed off with the runoff to the storm drainage system. Depending on the severity of the septic tank failure, the ponded water can have the characteristics of partially treated sanitary wastewater or nearly untreated sanitary wastewater. Thus, septic tank failures can contaminate the stormwater drainage system during both wet and dry weather.

Septic tank systems may fail even with good maintenance practices. Such failure can result when the soil is simply not permeable enough for the leaching field, or when the soil absorbance capacity is exceeded through long use. A tight clay soil may have such low permeability that the leaching capacity is very limited. If a number of homes are built in close proximity, their septic tank leaching fields may collectively exceed the soil's capacity, leading to a stormwater pollution problem. Even properly operating septic tank systems are a potential pollutant source. Because the basic function of the leaching field is to discharge partially treated effluent to the ground, this septic tank effluent can infiltrate into nearby stormwater drainage systems.

Various corrective methods exist for failing septic tank systems that pollute stormwater. These methods include: improve maintenance, institute preventative measures to avoid problems, and abandon the septic tank system with connections made to a sanitary sewerage system. In some cases, improved maintenance may be the answer. Some persons will not do any maintenance to their septic tank system until it fails (they note ponded water in the leaching field area). Then they call for the septic tank to be pumped. In many cases, this is not sufficient to correct the problem: it may be too little action too late. The preventative action of having the septic tank pumped should have taken place prior to failure of the system. Education may provide part of the remedy. The septic tank user may respond to exhortations to have the septic tank pumped on a regular basis, before failure. Coercion through ordinances may be another answer. Ordinances may require that the septic tank be pumped at a specified frequency, with a public body monitoring the program to ensure that maintenance has been carried out.

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It sometimes happens that soil conditions and population density rule out both voluntary or involuntary maintenance. In this case, it may be necessary to consider abandoning the septic tank system and installing a system consisting of sanitary sewers leading to a treatment plant. Another option consists of abandoning the septic tank treatment method in favor of small package treatment units that provide aerobic treatment of the sanitary wastewater which is then discharged to a regional leaching field. This option may succeed where the septic tank system has failed, because wastes treated in an aerobic unit may not have the leaching field clogging potential of wastes treated in an anaerobic septic tank. However, experience has shown that these advantages are only obtained with proper control and maintenance. Aerobic systems are more sensitive than conventional septic tank systems to improper maintenance and may therefore not offer any real benefits.

DIRECT SANITARY SEWERAGE CONNECTIONS

Due to indifference, ignorance, poor enforcement of ordinances, or other reasons, a stormwater drainage system may have sanitary wastewater sewerage direct connections. Obviously, the sanitary wastewater entering the storm drain will not receive any treatment and will pollute a large flow of stormwater, in addition to the receiving water. If the storm drain has a low dry-weather flow rate, the presence of sanitary wastewater may be obvious due to toilet paper, feces, and odors. In cases of high dry-weather flows, it may be more difficult to obviously detect raw sanitary wastewaters due to the low percentage of sanitary wastewater in the mixture. Even though the sanitary wastewater fraction may be low, the previously discussed field testing procedures (e.g., testing for surfactants, ammonia, potassium, and fluorides) will assist in the detection and quantification of sanitary wastewater contamination in the storm drainage system. Flow monitoring may show the variations in the flow rate that are typical of sanitary wastewater.

Dye testing can be effective in finding specific sanitary wastewater connections between a house and a storm drainage system. Dye, such as diluted rhodamine or fluorescein, is flushed down the toilet of a house and the storm drain is monitored to determine whether the dye appears. Care has to be exercised when using this method, as these dyes may stain fixtures that are being tested, and any spillage in the house causes stains that are very difficult to remove.

Monitoring of the storm drainage system with television cameras can show the locations of breaks in the storm drain where a sanitary wastewater sewer or house lateral was attached. Television cameras may also show discharges taking place at these locations, demonstrating that the lines are in active use.

Corrective measures involve undertaking a program of disconnecting the sanitary sewer connections to the storm drainage system and reconnecting them to a proper sanitary wastewater sewerage system. The storm drainage system then has to be repaired so that the holes left by the disconnected sanitary sewer entrances do not become a location for dirt and groundwater to enter.

REHABILITATING STORM OR SANITARY SEWERS TO ABATE CONTAMINATED WATER INFILTRATION

Infiltration of contaminated water into a stormwater drainage system can cause substantial pollution of the system. This could occur where a sanitary sewer overlies and crosses (or parallels) a storm drain, with sanitary wastewater exfiltrating from the sanitary sewer and percolating the storm drain. Other instances would be in areas of polluted groundwater, where the storm drainage is below the water table or intercepts infiltrating groundwater, or in areas having septic tank systems, as discussed previously.

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It would be best to correct the sanitary sewer if only one drainage system can be corrected. This would have the dual advantage of preventing infiltration of high or percolating groundwaters and preventing pollution of stormwater with exfiltrating sanitary wastewater. Rehabilitation of the drainage systems by use of inserted liners, or otherwise patching leaking areas, are possible corrective measures. It is important that all drains with infiltration problems be corrected for this corrective action to be effective. This would also include repairing house lateral sanitary wastewater lines, as well as the main drainage runs. However, these corrective measures are more likely to be cost effective when only a relatively small part of the complete drainage systems require rehabilitation.

ZONING AND ORDINANCES

Land use controls achieved by zoning have the potential to exacerbate problems or diminish them. For example, in an area with soils that are ill suited for septic tanks and leaching fields, the potential for future problems is increased if zoning allows small lots for single family residential development and allows septic tank systems. As the area develops, septic tank failures will become common, resulting in increased pollution of stormwater and groundwater. On the other hand, in areas having poor soils, zoning can require correspondingly larger lot sizes and larger leaching fields, resulting in fewer future problems. Ordinances may specify the results that have to be achieved by infiltration tests used to size leaching fields. Also, ordinances can require that a responsible public official be present when the infiltration test is run to decrease the likelihood of false or spurious results being reported. Certified septic tank installers, also checked by public official inspectors, should also be required to increase the likelihood of the system being installed correctly.

Zoning can also have a role to play in avoiding development of land that is subject to frequent flooding. In such land, flooding and high groundwater conditions can result in the sanitary sewerage system being gradually overloaded by infiltration so that cross flow to the storm drainage system can occur.

Ordinances can help to control problems by putting the force of law and public policy behind desirable practices. For example, ordinances can make mandatory practices such as septic tank maintenance that otherwise would be voluntary. By making the practice mandatory, desirable practices are performed on a regular schedule so that large problems have less opportunity to develop. Ordinances can also regulate the persons doing the pumping of septic tanks so that they discharge the septage to wastewater treatment plants where it can be properly treated rather than it being discharged improperly where the pollution problem is just transferred from one location to another.

Ordinances can also help prevent and or control pollution from many other sources by restrictions on: disposal of household toxic substances to storm drains, storage of chemicals by industry, disposal of industrial wash down water, etc.

Zoning and ordinances represent important means for governing bodies to anticipate problems, to avoid problems, and to manage problems, so that desirable ends are achieved and undesirable consequences are avoided. Enactment of zoning and ordinances occurs in the public arena where interested persons can participate and express their views and concerns. The public can become educated in this process, but zoning and ordinances have the desirable characteristic of being remembered and remaining enforceable long after an individual forgets, becomes disinterested, or becomes recalcitrant.

Another important step that municipalities can take is the development of policies and procedures for the management of spills from transportation (including both roadway and rail) and pipeline accidents. Spills should not be merely washed into the storm drainage system, but should be collected

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for proper treatment and disposal.

WIDESPREAD SANITARY SEWERAGE FAILURE

Connections (whether directly by piping or indirectly by exfiltration or infiltration) of sanitary sewers to the storm drainage system may be so widespread that the storm drainage system has to be recognized as a combined sewer system. This could also be the case when the prevalence of septic tank failures leads to widespread sanitary wastewater runoff to the storm drainage system. One usually thinks of a combined sewer system as having all of the sanitary sewer connections to the same sewers that carry stormwater, but the previous discussion suggests that there are degrees of a storm drainage system becoming a combined sewer system. Previously, the recommendations have been made that widespread failure of septic tank systems might necessitate the construction of a sanitary sewer to replace the septic tanks. Also recommended was a program of identifying and disconnecting sanitary sewers from the storm drainage system.

Prior to these actions taking place, the storm drainage system operates to some degree as a combined sewer system. It may be that the sanitary sewerage system is not capable of handling the load that would be imposed on it if a complete sewer separation program were undertaken. Or, in an extreme case, no sanitary sewer system may exist. By recognizing that a combined sewer system does in fact exist may help to focus attention on appropriate remedial measures. The resources may also not be available to undertake construction of a separate sanitary wastewater drainage system.

One should then focus on how to manage the combined sewer system that is in place. Management may require that end-of-pipe storage/treatment be investigated. Also, the combined sewer system may be tied into other combined sewers so that more centralized treatment and storage can be applied. Operation of a combined sewer system may be preferable to having the stormwater and the large number of sanitary entries receive no treatment.

An early identification and decision to designate a storm drainage system a combined sewer system, will prevent abortive time and costs being spent on further investigations. These resources can then be more effectively used to treat the newly designated combined sewer system.

In essence, recognition of a system as being a combined sewer system provides a focus in the regulatory community so that it may be possible to operate the system so as to minimize the damage to the environment.

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GLOSSARY

Accuracy - The combination of bias and precision of an analytical procedure which reflects the closeness of a measured value to a true value.

Baseflow - The dry-weather flow occurring in a drainage system, with no apparent source. Likely to be mostly infiltrating groundwaters in a sanitary or storm drainage system, but can also be contaminated with illicit wastewaters. See constant (or continual) dry-weather flow.

Batch dump - The disposal of a large volume of waste material during a short period of time. Usually an industrial waste.

Bias - A consistent deviation of measured values from the true value, caused by systematic errors in a procedure.

Coefficient of Variation (COV) - A measure of the spread of data (ratio of the standard deviation to the mean).

Combined Sewer - A sewer designed for receiving surface (dry- and wet-weather) runoff, municipal (sanitary and industrial) wastewater, and subsurface waters from infiltration. During dry weather, it acts as a sanitary sewer, but it also carries stormwater from wet-weather runoff.

Combined sewer overflow (CSO) - Flow from an outfall (discharge conduit) of a combined sewer collection system, in excess of the interceptor capacity or due to a malfunctioning or improperly set flow regulator, that is discharged into a receiving water and/or an auxiliary CSO control storage-treatment system.

Constant (or continual) dry-weather flow - Uninterrupted flow in a storm sewer or drainage ditch occurring in the absence of rain. See baseflow.

Deposits and stains - Any type of coating or discoloration that remains at an outfall as result of dry-weather discharges.

Detection limit - A number of different detection limits have been defined: IDL (instrument detection limit), is the constituent concentration that produces a signal greater than five times the signal to noise ratio of the instrument; MDL (method detection limit) is the constituent concentration that, when processed through a complete method, produces a signal with a 99 percent probability that it is different from a blank; PQL (practical quantification limit) is the lowest constituent concentration achievable among laboratories within specified limits during routine laboratory operations. The ratios of these limits are approximately: IDL:MDL:PQL = 1:4:20 (APHA, et al. 1989).

Direct (dry-weather) entries into the storm drainage system - Sources which enter a storm drainage system directly, usually by direct piping connections between the wastewater conduit and the storm drain.

Domestic sanitary wastewater - Sewage derived principally from human sources.

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Drainage area - The area of land from which a storm drainage system collects precipitation and storm runoff and then delivers the resulting stormwater to a specific point.

Dry-weather flow - Flow in a storm sewer or drainage ditch occurring in the absence of storm flow. But it is also a constituent of wet-weather flow. See baseflow.

Entries to storm drainage - Water (relatively clear or polluted) discharged into a stormwater drain from sources such as, but not limited to, direct industrial or sanitary wastewater connections, roof leaders, yard and area drains, cooling water connections, manhole covers, groundwater or subterraneous stormwater infiltration, etc.

Floatables - Floating materials, (plastic containers, condoms, sanitary napkins, tissues, corks, paper containers, wood, leaves, oil films, slimes, scum, etc.), that are either part of the inappropriate waste streams discharged to a stormwater system, or collected by flows which enter a stormwater drainage system.

Geographic Information System (GIS) - Computer software that maps land areas and produces images and information relating to the land area, e.g., topography, drainage, public utilities, roads, buildings, industry, land use, and demography.

Groundwater infiltration - Seepage of below water table groundwater and subterraneous stormwater into stormwater, sanitary wastewater, or combined sewer drainage systems, through such means as defective pipes, pipe joints, connections, or manhole walls.

Hardness - Caused by the presence of the divalent cations (principally calcium and magnesium) in water. Causes an increased amount of soap usage before producing a lather and scale to form in hot water pipes, boiler vessels, condensate return lines, cooling systems, kettles, etc.

House Lateral - A pipe connecting a house to a lateral or other sewerline. Also called a service connection.

Indirect dry-weather entries into the storm drainage system - Non-stormwater sources which enter a storm drainage system indirectly, usually by floor, areaway, and yard drains or inlets; and spills and dumping.

Industrial dry-weather entries into the storm drainage system - Any solid or liquid waste coming from industrial sources which enter storm drainage systems during periods of dry weather.

Infiltration - The process whereby water enters a drainage system underground through such means as defective pipes, pipe joints, connections, manhole walls, etc.

Inflow - The process whereby water enters a sanitary wastewater drainage system from surface locations, (e.g., through depressed manhole covers, yard and areaway inlets, roof leader setc.).

Intercepted stormwater/groundwater - The portion of surface runoff or groundwater moving through the soil that enters a storm drainage, combined sewer, or sanitary sewer system.

Interceptor - A sewer that receives flows from a number of wastewater trunk lines.

Intermittent dry-weather flow - Irregular flow in a storm drainage system occurring in the absence of storm flow.

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Lateral - A drain or sewer that has no other drains or sewers discharging into it, except for service connections, or house laterals.

Leaching field - A system which facilitates the infiltration of a septic tank effluent into the soil. This is typically done by a pipe and infiltrating trench system which takes the effluent from a septic tank and distributes it through the leaching field, where additional treatment of the effluent occurs as it percolates through the ground or soil column.

Monte Carlo probabilistic simulation - A statistical modeling approach used to determine the expected frequency and magnitude of an output by running repetitive simulations using statistically selected inputs for the model parameters.

Municipal sewage/wastewater - Sewage/wastewater from a community which may be composed of domestic sewage/wastewater, industrial wastewater and/or commercial wastewater, together with subsurface infiltration.

National Pollution Discharge Elimination System (NPDES) - A national system of permits issued to industrial, commercial, and municipal dischargers to limit the amount of pollutants that can be discharged to waters of the USA.

Non-contact cooling water - Water that decreases the temperature of an object, without ever physically contacting the object.

Nonpoint pollution source- Any unconfined and nondiscrete conveyance from which pollutants are discharged, or an urban drainage system not under the NPDES. These sources are usually from agricultural, silvicultural, and rural land areas..

Outfall - In this User's Guide, an outfall refers to a point at which a stormwater drainage system discharges to a receiving water. There is sometimes a concrete structure or retaining wall at this location to protect the end of the discharge pipe and prevent erosion of the receiving water bank.

Pathogen - A disease-causing microorganism.

Point source - Any discernible, confined, and discrete conveyance from which pollutants are, or may be, discharged. Under the NPDES it is an outfall discharge, or overflow of treated or untreated sanitary, industrial, combined sewage, or stormwater (from a municipality greater than 100,000 in population).

Pollutant - Any material in water or wastewater interfering with designated beneficial uses.

Potable water - Water that has been treated, or is naturally fit for drinking, i.e., the water has no harmful contents to make it unsuitable for human consumption.

Precision - The measure of the degree of agreement among replicate analyses of a sample, usually expressed as the standard deviation.

Pretreatment - The removal of material such as, gross solids, grit, grease, metals, toxicants. etc. or treatment such as aeration, pH adjustment, etc. to improve the quality of a wastewater prior to discharge to a municipal wastewater system. This is usually done by the industrial user of the water, but can also refer to the initial treatment processes of a sewage treatment plant.

Process line discharge - The disposal of anything used in, or resulting from, a manufacturing process.

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Process water - Water used in industry to perform a variety of functions, or as an actual product ingredient.

Receiving waters - Natural or man-made water systems into which stormwaters, or wastewaters, are discharged.

Rinse water - Water that cleans or reduces the temperature of an object through actual physical contact with the object.

Sanitary sewer - A sanitary wastewater drainage system intended to carry wastewaters from residences, commercial buildings, industrial plants, and institutions together with minor quantities of groundwater, stormwater and surface water that are not admitted intentionally [40 CFR 35.2005 (b) (37)].

Sanitary wastewater - Wastewater of human origin.

Service Connection - See house lateral

Septic tank - A tank which receives sanitary wastewater direct from its source, (usually residential), and permits settling of the heavy solids and floatation of greases and fats along with anaerobic digestion. Septic tanks, typically need to meet minimum regulatory standards, e.g., minimum volume and detention time.

Sewage - In this text the term "sewage" refers to sanitary wastewater or wastewaters generated from commercial or industrial operations, it does not include stormwater.

Sewer - A pipe, conduit or drain generally closed, but normally not flowing full, for carrying sanitary, industrial and commercial wastewater and storm-induced (combined wastewater and stormwater) flows.

Sewerage - System of piping and appurtenances, with and without control-treatment facilities for collecting and conveying wastewaters with or without pollution abatement from source to discharge.

Specific Conductivity - Expressed in microSiemens/cm (or micromhos/cm). It is an indication of the dissolved solids (charged) concentration in a liquid.

Storm drainage discharge - Flow from a storm drain that is discharged to a receiving water.

Storm drain - A pipe, or natural or man-made channel, or ditch, that is designed to carry only stormwater, surface runoff, street washwaters, and drainage from source to point of discharge [40 CFR 35.2005 (b) (47)].

Stormwater - Water resulting from precipitation which either infiltrates into the ground, impounds/puddles, and/or runs freely from the surface, or is captured by storm drainage, a combined sewer, and to a limited degree, by sanitary sewer facilities. See urban runoff and urban stormwater runoff.

Surfactants - Surface-active agents and common components in detergents which affect the surface tension of water and can cause foaming.

SIC - Standard Industrial Classification, a code used to describe an industry.

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Total solids - The entire quantity of solids in the liquid flow or volume including the dissolved and particulate (suspended, floatable, and settleable) fractions.

Toxicity - The degree to which a pollutant causes physiological harm to the health of an organism.

Tracer - In this User's Guide, a tracer is a distinct component, or combination of components ("fingerprint"), of a polluting source which is identified in order to confirm the entry of the polluting source to a storm drainage system.

Trace Metals - Metals present in small concentrations. From a regulatory standpoint, this usually refers to metal concentrations that can cause toxicity at trace concentrations.

Turbidity - The lack of clarity in the water usually caused by suspended particulate matter and measured by interference to light penetration.

Urban runoff - Any runoff stormwater from an urban drainage area that reaches a receiving water body or subsurface. During dry weather, it may be comprised of many baseflow components, both relatively uncontaminated and contaminated. See stormwater and urban stormwater runoff.

Urban stormwater runoff - Stormwater from an urban drainage area that reaches a receiving water body or subsurface caused by weather precipitation (rain, snow, etc.). See stormwater and urban runoff.

Watershed - A geographic region (area of land) within which precipitation drains into a particular river, drainage system or body of water that has one specific delivery point.

Wet-weather flow - Any flow resulting from precipitation (rain, snow, etc.) which may introduce contaminants into storm drainage combined sewerage, or sanitary sewerage systems.

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*U.S GOVERNMENT PRINTING OFFICE:1993-750-002/60112

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Investigation of Inappropriate Pollutant Entries into Storm Drainage Systems

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Project Summary

Potential Groundwater Contamination from Intentional and Nonintentional Stormwater Infiltration

Robert Pitt, Shirley Clark, and Keith Parmer

The research summarized here was conducted during the first year of a 3-yr cooperative agreement to identify and control stormwater toxicants, especially those adversely affecting groundwater. The purpose of this research effort was to review the groundwater contamination literature as it relates to stormwater. Potential problem pollutants were identified, based on their mobility through the unsaturated soil zone above groundwater, their abundance in stormwater, and their treatability before discharge. This information was used with earlier EPA research results to identify the possible sources of these potential problem pollutants. Recommendations were also made for stormwater infiltration guidelines in different areas and monitoring that should be conducted to evaluate a specific stormwater for its potential to contaminate groundwater.

This Project Summary was developed by EPA's Risk Reduction Engineering Laboratory, Cincinnati, OH, to announce key findings of the research project that is fully documented in a separate report of the same title (see Project Report ordering information at back).

Introduction

Before urbanization, groundwater was recharged by precipitation infiltrating through pervious surfaces, including grasslands and woods. This infiltrating water was relatively uncontaminated. Urbanization, however, reduced the permeable soil surface area through which recharge by infiltration could occur. This resulted in much less groundwater recharge and greatly increased surface runoff. In addition, the waters available for recharge generally carried increased quantities of pollutants. With urbanization, waters having elevated contaminant concentrations also recharge groundwater, including effluent from domestic septic tanks, wastewater from percolation basins and industrial waste injection wells, infiltrating stormwater, and infiltrating water from agricultural irrigation. This report addresses potential groundwater problems associated with stormwater toxicants and describes how conventional stormwater control practices can reduce these problems.

Sources of Pollutants

High bacteria populations have been found in sheetflow samples from sidewalks, roads, and some bare ground (collected from locations where dogs would most likely be "walked"). Tables 1 and 2 summarize toxicant concentrations and likely sources or locations having some of the highest concentrations found during an earlier phase of this EPA-funded research. The detection frequencies for the heavy metals are all close to 100% for all source areas, and the detection frequencies for the organics listed on these tables ranged from about 10% to 25%. Vehicle service areas had the greatest abundance of observed organics.

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Table 1. Concentrations of Heavy Metals in Observed Areas (µg/L)

Toxicant	Highest Median		Highest Observed	
Cadmium	Vehicle service area runoff	8	Street runoff	220
Chromium	Landscaped area runoff	100	Roof runoff	510
Copper	Urban receiving water	160	Street runoff	1250
Lead	CSO	75	Storage area runoff	930
Nickel	Parking area runoff	40	Landscaped area runoff	130
Zinc	Roof runoff	100	Roof runoff	1580

Table 2. Maximum Concentrations of Toxic Organics from Observed Sources

Toxicant	Maximum, µg/L	Detection Frequency, %	Significant Sources
Benzo (a) anthracene	60	12	Gasoline, wood preservative
Benzo (b) fluoranthene	226	17	Gasoline, motor oils
Benzo (k) fluoranthene	221	17	Gasoline, bitumen, oils
Benzo (a) pyrene	300	17	Asphalt, gasoline, oils
Fluoranthene	128	23	Oils, gasoline, wood preservative
Naphthalene	296	13	Coal tar, gasoline, insecticides
Phenanthrene	69	10	Oils, gasoline, coal tar
Pyrene	102	19	Oils, gasoline, bitumen, coal tar, wood preservative
Chlordane	2.2	13	Insecticide
Butyl benzyl phthalate	128	12	Plasticizer
Bis (2-chloroethyl) ether	204	14	Fumigant, solvents, insecticides, paints, lacquers, varnishes
Bis (2-chloroisopropyl) ether	217	14	Pesticide manufacturing
1,3-Dichlorobenzene	120	23	Pesticide manufacturing

Stormwater Constituents Having High Potential to Contaminate Groundwater

Nutrients

Nitrates are one of the most frequently encountered contaminants in groundwater. Phosphorus contamination of groundwater has not been as widespread, or as severe, as that of nitrogen compounds. Whenever nitrogen-containing compounds come into contact with soil, a potential exists for nitrate leaching into groundwater, especially in rapid-infiltration wastewater basins, stormwater infiltration devices, and agricultural areas. Nitrate has leached from fertilizers and affected groundwaters under various turf grasses in urban areas, including golf courses, parks, and home lawns. Significant leaching of nitrates occurs during the cool, wet seasons. Cool temperatures reduce denitrification and ammonia volatilization and limit microbial nitrogen immobilization and plant uptake. The use of slow-release fertilizers (including composted organic mulches, urea formaldehyde (UF), methylene urea, isobutylidene diurea (IBDU), and sulfur-coated urea) is recommended in areas having potential groundwater nitrate problems.

Residual concentrations of nitrate in soil vary greatly and depend on the soil texture, mineralization, rainfall and irrigation patterns, organic matter content, crop yield, nitrogen fertilizer/sludge application rate, denitrification, and soil compaction. Nitrate is highly soluble (>1 kg/L) and will stay in solution in the percolation water. If it leaves the root zone without being taken-up by plants, it will readily reach the groundwater.

Pesticides

Urban pesticide contamination of groundwater can result from municipal and homeowner use for pest control and the subsequent collection of the pesticide in stormwater runoff. Pesticides that have been found in urban groundwaters include: 2,4-D, 2,4,5-T, atrazine, chlordane, diazinon, ethion, malathion, methyl trithion, silvex, and simazine. Heavy repetitive use of mobile pesticides (those that are not likely to be retained by various processes in the soil before they reach the groundwater, such as 2,4-D, acenaphthylene, alachlor, atrazine, cyanazine, dacthal, diazinon, dicamba, and malathion) on irrigated and sandy soils will likely contaminate groundwater. Fungicides and nematocides must be mobile to reach the target pest, and hence, they generally have the highest groundwater contamination potential. Pesticide leaching depends on patterns of use, soil texture, total organic carbon content of the soil, pesticide persistence, and depth to the water table.

The greatest pesticide mobility occurs in areas with coarse-grained or sandy soils without a hardpan layer, and with soils that have low clay and organic matter content and high permeability. Structural voids, generally found in the surface layer of finer-textured soils rich in clay, can transmit pesticides rapidly when the voids are filled with water and the adsorbing surfaces of the soil matrix are bypassed. In general, pesticides with low water solubilities, high octanol-water partitioning coefficients, and high carbon partitioning coefficients are less mobile. The slower moving pesticides that may better sorb to soils have been recommended for use in areas of groundwater contamination concern. These include the fungicides iprodione and triadimefon, the insecticides isofenphos and chlorpyrifos, and the herbicide glyphosate.

Pesticides decompose in soil and water, but the total decomposition time can range from days to years. Literature half-lives for pesticides generally apply to surface soils and do not account for the reduced microbial activity found deep in the vadose zone. Pesticides with a 30-day half life can show considerable leaching. An order-of-magnitude difference in half-life results in a five- to ten-fold difference in percolation loss. Organophosphate pesticides are less persistent than organochlorine pesticides, but they also are not strongly adsorbed by the sediment and are likely to leach into the vadose zone and the groundwater.

Other Organics

The most commonly occurring organic compounds found in urban groundwaters include phthalate esters (especially bis(2-ethylhexyl)phthalate) and phenolic compounds. Other, more rarely found, organics include the volatiles: benzene, chloroform, methylene chloride, trichloroethylene, tetrachloroethylene, toluene, and xylene. Polycyclic aromatic hydrocarbons (PAHs) (especially benzo(a)anthracene, chrysene, anthracene, and benzo(b)fluoroanthenene) have also been found in groundwaters near industrial sites.

Groundwater contamination from organics, like that from other pollutants, occurs

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more readily in areas with sandy soils and where the water table is near the land surface. Organics can be removed from the soil and recharge water by volatilization, sorption, and degradation. Volatilization can significantly reduce the concentrations of the most volatile compounds in groundwater, but the rate of gas transfer from the soil to the air is usually limited by the presence of soil water. Hydrophobic sorption onto soil organic matter limits the mobility of less soluble base/neutral and acid extractable compounds through organic soils and the vadose zone. Sorption is not always a permanent removal mechanism, however. Organic resolubilization can occur during wet periods following dry periods. Many organics can be degraded by microorganisms, at least partially, but others cannot. Temperature, pH, moisture content, ion exchange capacity of the soil, and air availability may limit the microbial degradation potential for even the most degradable organic compound.

Microorganisms

Viruses have been detected in groundwater where stormwater recharge basins were located short distances above the aquifer. Enteric viruses are more resistant to environmental factors than are enteric bacteria, and they exhibit longer survival times in natural waters. They can occur in potable and marine waters in the absence of fecal coliforms. Enteroviruses are also more resistant to commonly used disinfectants than are indicator bacteria (such as fecal coliforms), and they can occur in groundwater in the absence of indicator bacteria.

The factors that affect the survival of enteric bacteria and viruses in the soil include pH, antagonism from soil microflora, moisture content, temperature, sunlight, and organic matter. The two most important attributes of viruses that permit their long-term survival in the environment are their structure and very small size. These characteristics permit virus occlusion and protection within colloid-size particles. Viral adsorption is promoted by increasing cation concentration, decreasing pH, and decreasing soluble organics. Since the movement of viruses through soil to groundwater occurs in the liquid phase and involves water movement and associated suspended virus particles, the distribution of viruses between the adsorbed and liquid phases determines the viral mass available for movement. Once the virus reaches the groundwater, it can travel laterally through the aquifer until it is either adsorbed or inactivated.

The major bacterial removal mechanisms in soil are straining at the soil surface and at intergrain contacts, sedimentation, sorption by soil particles, and inactivation. Because their size is larger than viruses, most bacteria are retained near the soil surface because of this straining effect. In general, enteric bacteria survive in soil between 2 and 3 mo, although survival times up to 5 yr have been documented.

Metals

From a groundwater pollution standpoint, the metals in stormwater presenting the most environmental concern are aluminum, arsenic, cadmium, chromium, copper, iron, lead, mercury, nickel, and zinc. The majority of these metals (with the common exception of zinc) are, however, mostly associated with the particulate fractions and can be mostly removed by either sedimentation or filtration processes.

In general, studies of recharge basins receiving large metal loads found that most of the heavy metals are removed either in the basin sediment or in the vadose zone. Dissolved metal ions are removed from stormwater during infiltration mostly by adsorption onto the near-surface particles in the vadose zone, and the particulate metals are filtered out at the soil surface. Studies at recharge basins found that lead, zinc, cadmium, and copper accumulated at the soil surface with little downward movement over many years. At a commercial site, however, nickel, chromium, and zinc concentrations have exceeded regulatory limits in the soils below a recharge area. Allowing percolation ponds to go dry between storms can be counterproductive to the removal of lead from the water during recharge. Apparently, the adsorption bonds between the sediments and the metals can be weakened during the drying period.

Similarities in water quality between runoff water and groundwater have shown that there is significant downward movement of copper and iron in sandy and loamy soils. Arsenic, nickel, and lead, however, did not significantly move downward through the soil to the groundwater. The exception to this was some downward movement of lead with the percolation water in sandy soils beneath stormwater recharge basins. Zinc, which is more soluble than iron, has been found in higher concentrations in groundwater than iron. The order of attenuation in the vadose zone from infiltrating stormwater is: zinc (most mobile) > lead > cadmium > manganese > copper > iron > chromium > nickel > aluminum (least mobile).

Salts

Salt applications for winter traffic safety is a common practice in many northern areas, and the sodium and chloride, which are collected in the snowmelt, travel down through the vadose zone to the groundwater with little attenuation. Soil is not very effective at removing salts. Salts that are still in the percolation water after it travels through the vadose zone will contaminate the groundwater. Infiltrating stormwater has increased sodium and chloride concentrations above background concentrations. Fertilizer and pesticide salts also accumulate in urban areas and can leach through the soil to the groundwater.

Studies of depth of pollutant penetration in soil have shown that sulfate and potassium concentrations decrease with depth, whereas sodium, calcium, bicarbonate, and chloride concentrations increase with depth. Once contamination with salts begins, the movement of salts into the groundwater can be rapid. The salt concentration may not lessen until the source of the salts is removed.

Treatment of Stormwater

Table 3 summarizes the filterable fraction of toxicants found in runoff sheet flows from many urban areas found during an earlier phase of this EPA-funded research. Pollutants that are mostly in filterable forms have a greater potential of affecting groundwater and are more difficult to control with the use of conventional stormwater control practices which mostly rely on sedimentation and filtration principles. Luckily, most of the toxic organics and metals are associated with the nonfilterable (suspended solids) fraction of the wastewaters during wet weather. Possible exceptions include zinc, fluoranthene, pyrene, and 1,3-dichlorobenzene, which may be mostly found in the filtered sample portions. Pollutants in dry-weather storm drainage flows, however, tend to be much more associated with filtered sample fractions and would not be as readily controlled with the use of sedimentation.

Sedimentation is the most common fate and control mechanism for particulate-related pollutants. This would be common for most stormwater pollutants, as noted above. Particulate removal can occur in many conventional stormwater control processes, including catchbasins, screens, drainage systems, and detention ponds. Sorption of pollutants onto solids and metal precipitation increases the sedimentation potential of these pollutants and also encourages more efficient bonding of the pollutants in soils to prevent their leaching

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Table 3. Reported Filterable Fractions of Stormwater Toxicants from Source Areas

Constituent	Filterable Fraction (%)
Cadmium	20 to 50
Chromium	<10
Copper	<20
Iron	Small amount
Lead	<20
Nickel	Small amount
Zinc	>50
Benzo (a) anthracene	None found in filtered fraction
Fluoranthene	65
Naphthalene	25
Phenanthrene	None found in filtered fraction
Pyrene	95
Chlordane	None found in filtered fraction
Butyl benzyl phthalate	Irregular
Bis (2-chloroethyl) ether	Irregular
Bis (2-chlorisopropyl) ether	None found in filtered fraction
1,3-Dichlorobenzene	75

to groundwaters. Detention ponds are probably the most common management practice for the control of stormwater runoff. If properly designed, constructed, and maintained, wet detention ponds can be very effective in controlling a wide range of pollutants. The monitored performance of wet detention ponds indicates more than 90% removal for suspended solids, 70% for BOD_5 and COD, about 60% to 70% for nutrients, and about 60% to 95% for heavy metals. Catchbasins are very small sedimentation devices. Adequate cleaning can help reduce the total solids and lead urban runoff yields by between 10% and 25%, and COD, total Kjeldahl nitrogen, total phosphorus, and zinc by between 5% and 10%. Other important fate mechanisms available in wet detention ponds, but which are probably not important in small enclosed sump devices such as catchbasins, include volatilization and photolysis. Biodegradation, biotransformation, and bioaccumulation (into plants and animals) may also occur in larger and open ponds.

Upland infiltration devices (such as infiltration trenches, porous pavements, percolation ponds, and grass roadside drainage swales) are located at urban source areas. Infiltration (percolation) ponds are usually located at stormwater outfalls or at large paved areas. These basins, along with perforated storm sewers, can infiltrate flows and pollutants from all upland sources combined. Infiltration devices can safely deliver large fractions of the surface flows to groundwater, if carefully designed and located. Local conditions that can make stormwater infiltration inappropriate include steep slopes, slowly percolating soils, shallow groundwater, and nearby groundwater uses.

Grass filter strips may be quite effective in removing particulate pollutants from overland flows. The filtering effects of grasses, along with increased infiltration/recharge, reduce the particulate sediment load from urban landscaped areas. Grass swales are another type of infiltration device and can be used in place of curb and gutter drainages in most land uses, except possibly strip commercial and high density residential areas. Grass swales allow the recharge of significant amounts of surface flows. Swales can also reduce pollutant concentrations because of filtration. Soluble and particulate heavy metal (copper, lead, zinc, and cadmium) concentrations can be reduced by at least 50%, COD, nitrate nitrogen, and ammonia nitrogen concentrations can be reduced by about 25%, but only inconsistent concentration reductions can be expected for organic nitrogen, phosphorus, and bacteria.

Sorption of pollutants to soils is probably the most significant fate mechanism of toxicants in biofiltration devices. Many of the devices also use sedimentation and filtration to remove the particulate forms of the pollutants from the water. Incorporation of the pollutants onto soil with subsequent biodegradation and minimal leaching to the groundwater is desired. Volatilization, photolysis, biotransformation, and bioconcentration may also be significant in grass filter strips and grass swales. Underground seepage drains and porous pavements offer little biological activity to reduce toxicants.

Results and Conclusions

This entire research project will provide guidance on critical source area treatment, especially for the protection of groundwater quality. Much of the information will also be useful for analyzing stormwater problems and needed controls for surface water discharges.

Table 4 is a summary of the pollutants found in stormwater that may cause groundwater contamination problems for various reasons. This table does not consider the risk associated with using groundwater contaminated with these pollutants. Causes of concern include high mobility (low sorption potential) in the vadose zone, high abundance (high concentrations and high detection frequencies) in stormwater, and high soluble fractions (small fraction associated with particulates that would have little removal potential using conventional stormwater sedimentation controls) in the stormwater. The contamination potential is the lowest rating of the influencing factors. As an example, when no pretreatment is used before percolation through surface soils, the mobility and abundance criteria are most important. When a compound is mobile but in low abundance (such as for volatile organic compounds, VOCs), then the groundwater contamination potential would be low. When the compound is mobile, however, and also in high abundance (such as for sodium chloride, in certain conditions), then the groundwater contamination potential would be high. When sedimentation pretreatment is to be used before infiltration, then some of the pollutants will likely be removed before infiltration. In this case, all three influencing factors (pollutant mobility, pollutant abundance in stormwater, and fraction of the pollutant associated with the filtered sample fraction) would be considered. As an example, chlordane would have a low contamination potential with sedimentation pretreatment, whereas it would have a moderate contamination potential when no pretreatment is used. In addition, when subsurface infiltration/injection is used instead of surface percolation, the compounds would most likely be more mobile, making the abundance criteria the most important, with some regard given to the filterable fraction information for operational considerations.

This table is only appropriate for initial estimates of contamination potential because of the simplifying assumptions made, such as the worst case mobility conditions assumed (for sandy soils having low organic content). When the soil is clayey and has a high organic content, then most of the organic compounds would be less mobile than that shown on this table. The abundance and filterable fraction information is generally applicable for warm weather stormwater runoff in residential and commercial areas. The pollutant concentrations and detection

frequencies, however, would be greater for critical source areas (especially vehicle service areas) and critical land uses (especially manufacturing industrial areas).

The stormwater pollutants of most concern (those that may have the greatest adverse impacts on groundwaters) include:

- Nutrients: nitrate has a low to moderate potential for contaminating groundwater when both surface percolation and subsurface infiltration/injection are used because of its relatively low concentrations in most stormwaters. When the stormwater nitrate concentration is high, then the groundwater contamination potential would likely also be high.
- Pesticides: lindane and chlordane have moderate potentials for contaminating groundwater when surface percolation (with no pretreatment) or when subsurface injection (with minimal pretreatment) are used. The groundwater contamination potentials for both of these compounds would very likely be substantially reduced with adequate sedimentation pretreatment.
- Other organics: 1,3-dichlorobenzene may have a high potential for contaminating groundwater when subsurface infiltration/injection (with minimal pretreatment) is used. It would, however, probably have a lower groundwater contamination potential for most surface percolation practices because of its relatively strong sorption to vadose zone soils. Both pyrene and fluoranthene would also very likely have high groundwater contamination potentials for subsurface infiltration/injection practices, but lower contamination potentials for surface percolation practices because of their more limited mobility through the unsaturated zone (vadose zone). Others (including benzo(a)anthracene, bis (2-ethylhexyl) phthalate, pentachlorophenol, and phenanthrene) may also have moderate groundwater contamination potentials when surface percolation with no pretreatment, or subsurface injection/infiltration, is used. These compounds would have low groundwater contamination potentials when surface infiltration is used with sedimentation pretreatment. VOCs may also have high groundwater contamination potentials if present in the stormwater (which is possible for some industrial and commercial facilities and vehicle service establishments).
- Pathogens: enteroviruses very likely have high potentials for contaminating groundwater when any percolation or subsurface infiltration/injection practice is used, depending on their presence in stormwater (especially if contaminated with sanitary sewage). Other pathogens, including *Shigella*, *Pseudomonas aeruginosa*, and various protozoa, would also have high groundwater contamination potentials when subsurface infiltration/injection practices are used without disinfection. When disinfection (especially by chlorine or ozone) is used, then disinfection by-products (such as trihalomethanes or ozonated bromides) would have high groundwater contamination potentials.
- Heavy Metals: nickel and zinc possibly have high potentials for contaminating groundwater when subsurface infiltration/injection is used. Chromium and lead would have moderate groundwater contamination potentials for subsurface infiltration/injection practices. All metals would possibly have low groundwater contamination potentials when surface infiltration is used with sedimentation pretreatment.
- Salts: chloride would very likely have a high potential for contaminating groundwater in northern areas where road salts are used for traffic safety, irrespective of the pretreatment, infiltration, or percolation practices used.

Pesticides have been mostly found in urban runoff from residential areas, especially in dry weather flows associated with landscaping irrigation runoff. The other organics, especially the volatiles, are mostly found in industrial areas. The phthalates are found in all areas. The PAHs are also found in runoff from all areas, but they are in higher concentrations and occur more frequently in industrial areas. Pathogens are most likely associated with sanitary sewage contamination of storm drainage systems, but several bacterial pathogens are commonly found in surface runoff in residential areas. Zinc is mostly found in roof runoff and other areas where galvanized metal comes into contact with rainwater. Salts are at their greatest concentrations in snowmelt and early spring runoff in northern areas.

The control of these compounds requires various approaches, including source area controls, end-of-pipe controls, and pollution prevention. All dry weather flows should be diverted from infiltration devices because of their potentially high concentrations of soluble heavy metals, pesticides, and pathogens. Similarly, all runoff from manufacturing industrial areas should also be diverted from infiltration devices because of their relatively high concentrations of soluble toxicants. Combined sewer overflows should also be diverted because of sewage contamination. In areas of snow and ice control, winter snowmelt and runoff and early spring runoff should also be diverted from infiltration devices.

All other runoff should include pretreatment using sedimentation processes before infiltration, to both minimize groundwater contamination and to prolong the life of the infiltration device (if needed). This pretreatment can take the form of grass filters, sediment sumps, wet detention ponds, etc., depending on the runoff volume to be treated, treatment flow rate, and other site specific factors. Pollution prevention can also play an important role in minimizing groundwater contamination problems, including reducing the use of galvanized metals, pesticides, and fertilizers in critical areas. The use of specialized treatment devices, such as those being developed and tested during this research, can also play an important role in treating runoff from critical source areas before these more contaminated flows commingle with cleaner runoff from other areas. Sophisticated treatment schemes, especially the use of chemical processes or disinfection, may not be warranted, except in special cases, especially when the potential of forming harmful treatment by-products (such as THMs and soluble aluminum) is considered.

The use of surface percolation devices (such as grass swales and percolation ponds) that have a substantial depth of underlying soils above the groundwater is preferable to the use of subsurface infiltration devices (such as dry wells, trenches or seepage drains, and especially injection wells), unless the runoff water is known to be relatively free of pollutants. Surface devices are able to take greater advantage of natural soil pollutant removal processes. Unless all percolation devices are carefully designed and maintained, however, they may not function properly and may lead to premature hydraulic failure or contamination of the groundwater.

Recommendations

With a reasonable degree of site-specific design considerations to compensate for soil characteristics, infiltration may be

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Table 4. Potential of Stormwater Pollutants to Contaminate Groundwater

Compounds		Mobility (sandy/low organic soils)	Abundance in Stormwater	Fraction Filterable	Contamination Potential: Surface Infilt. and No Pretreatment	Contamination Potential: Surface Infilt. with Sedimentation	Contamination Potential: Sub-surface Inj. with Minimal Pretreatment
Nutrients	nitrates	mobile	low/moderate	high	low/moderate	low/moderate	low/moderate
Pesticides	2,4-D	mobile	low	likely low	low	low	low
	γ-BHC (lindane)	intermediate	moderate	likely low	moderate	low	moderate
	malathion	mobile	low	likely low	low	low	low
	atrazine	mobile	low	likely low	low	low	low
	chlordane	intermediate	moderate	very low	moderate	low	moderate
	diazinon	mobile	low	likely low	low	low	low
Other organics	VOCs	mobile	low	very high	low	low	low
	1,3-dichloro-benzene	low	high	high	low	low	high
	anthracene	intermediate	low	moderate	low	low	low
	benzo(a) anthracene	intermediate	moderate	very low	moderate	low	moderate
	bis (2-ethylhexyl) phthalate	intermediate	moderate	likely low	moderate	low	moderate
	butyl benzyl phthalate	low	low/moderate	moderate	low	low	low/moderate
	fluoranthene	intermediate	high	high	moderate	moderate	high
	fluorene	intermediate	low	likely low	low	low	low
	naphthalene	low/inter.	low	moderate	low	low	low
	pentachlorophenol	intermediate	moderate	likely low	moderate	low	moderate
	phenanthrene	intermediate	moderate	very low	moderate	low	moderate
	pyrene	intermediate	high	high	moderate	moderate	high
Pathogens	enteroviruses	mobile	likely present	high	high	high	high
	Shigella	low/inter.	likely present	moderate	low/moderate	low/moderate	high
	Pseudomonas aeruginosa	low/inter.	very high	moderate	low/moderate	low/moderate	high
	protozoa	low/inter.	likely present	moderate	low/moderate	low/moderate	high
Heavy metals	nickel	low	high	low	low	low	high
	cadmium	low	low	moderate	low	low	low
	chromium	inter./very low	moderate	very low	low/moderate	low	moderate
	lead	very low	moderate	very low	low	low	moderate
	zinc	low/very low	high	high	low	low	high
Salts	chloride	mobile	seasonally high	high	high	high	high

very effective in controlling both urban runoff quality and quantity problems. This strategy encourages infiltration of urban runoff to replace the natural infiltration capacity lost through urbanization and to use the natural filtering and sorption capacity of soils to remove pollutants; however, the potential for some types of urban runoff to contaminate groundwater through infiltration requires some restrictions. Infiltration of urban runoff having potentially high concentrations of pollutants that may pollute groundwater requires adequate pretreatment or the diversion of these waters away from infiltration devices. The following general guidelines for the infiltration of stormwater and other storm drainage effluent are recommended in the absence of comprehensive site-specific evaluations:

- Dry weather storm drainage effluent should be diverted from infiltration devices because of their probable high concentrations of soluble heavy metals, pesticides, and pathogenic microorganisms.
- Combined sewage overflows should be diverted from infiltration devices because of their poor water quality, especially their high pathogenic microorganism concentrations and high clogging potential.
- Snowmelt runoff should be diverted from infiltration devices because of its potential for having high concentrations of soluble salts.
- Runoff from manufacturing industrial areas should be diverted from infiltration devices because of its potential for having high concentrations of soluble toxicants.
- Construction site runoff must be diverted from stormwater infiltration devices (especially subsurface devices) because of its high suspended solids concentrations, which would quickly clog infiltration devices.
- Runoff from other critical source areas, such as vehicle service facilities and large parking areas, should at least receive adequate pretreatment to eliminate their groundwater contamination potential before infiltration.
- Runoff from residential areas (the largest component of urban runoff in most cities) is generally the least polluted urban runoff flow and should be considered for infiltration. Very little treat-

ment of residential area stormwater runoff should be needed before infiltration, especially if surface infiltration is through the use of grass swales. When subsurface infiltration (seepage drains, infiltration trenches, dry wells, etc.) is used, then some pretreatment may be needed, such as by using grass filter strips, or other surface filtration devices.

Recommended Stormwater Quality Monitoring to Evaluate Potential Groundwater Contamination

Most past stormwater quality monitoring efforts have not adequately evaluated stormwater's potential for contaminating groundwater. The following list shows the stormwater contaminants that are recommended for monitoring when stormwater contamination potential needs to be considered, or when infiltration devices are to be used. Other analyses are appropriate for additional monitoring objectives (such as evaluating surface water problems). In addition, all phases of urban runoff should be sampled, including stormwater runoff, dry-weather flows, and snowmelts.

- Urban runoff contaminates with the potential to adversely affect groundwater:
  - Nutrients (especially nitrates)
  - Salts (especially chloride)
  - VOCs (if expected in the runoff, such as runoff from manufacturing industrial or vehicle service areas, could screen for VOCs with purgable organic carbon analyses)
  - Pathogens (especially enteroviruses, if possible, along with other pathogens such as *Pseudomonas aeruginosa*, *Shigella*, and pathogenic protozoa)
  - Bromide and total organic carbon (to estimate disinfection by-product generation potential, if disinfection by either chlorination or ozone is being considered)
  - Pesticides, in both filterable and total sample components (especially lindane and chlordane)
  - Other organics, in both filterable and total sample components (especially 1,3 dichlorobenzene, pyrene, fluoranthene, benzo(a)anthracene, bis (2-ethylhexyl) phthalate, pentachlorophenol, and phenanthrene)
  - Heavy metals, in both filterable and total sample components (especially chromium, lead, nickel, and zinc)
- Urban runoff compounds with the potential to adversely affect infiltration and injection operations:
  - Sodium, calcium, and magnesium (to calculate the sodium adsorption ratio to predict clogging of clay soils)
  - Suspended solids (to determine the need for sedimentation pretreatment to prevent clogging)

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