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November 12, 2015
CIWQS Place ID 818597 (SG)
Regulatory Measure ID 403119

Santa Clara Valley Water District
5750 Almaden Expressway
San Jose, CA 95118-3686

Attention: Mr. James Manitakos
Email: JManitakos@valleywater.org

Subject: Comments on the Draft Environmental Impact Report for Upper Berryessa Creek Flood Risk Management Project, Santa Clara County, SCH No. 2001104013

Dear Mr. Manitakos:

San Francisco Bay Regional Water Quality Control Board (Water Board) staff has reviewed the *Public Review Draft Environmental Impact Report for the Upper Berryessa Creek Flood Risk Management Project (State Clearinghouse No. 2001104013)* (DEIR) prepared by the Santa Clara Valley Water District (District) pursuant to the California Environmental Quality Act (CEQA). The project purpose is to convey the 1 percent exceedance probability flood event in Berryessa Creek from U.S. Interstate 680 in the City of San Jose for 2.2 miles downstream to Calaveras Boulevard in the City of Milpitas (Project).

The District is the local sponsor for the Project that the U.S. Army Corps of Engineers is constructing. The District is contributing a significant portion of the project cost; managing all real estate transactions for right-of-way land acquisition and easements; and will own and operate the project after it is constructed. Although the Corps previously screened alternatives in the *General Reauthorization Report/Environmental Impact Statement* (GRR/EIS) (March 2014), the District must also analyze alternatives pursuant to CEQA. The Corps-selected project design includes (but is not limited to) a roughly 1,300 foot long, 1.5 foot high floodwall. The District's preferred alternative is the same as the Corps' but with modifications which increase the length of the floodwall to about 2,200 feet, and the height by up to 0.5 feet. The added length and height would bring Alternative 2A to meet the Federal Emergency Management Administration's (FEMA) standards. As described further below, we provide the following comments on the DEIR, including, but not limited to:

- The DEIR alternatives analysis is limited to that of the Corps' GRR/EIS, so does not meet CEQA requirements to include a full array of feasible alternatives.

- Inconsistencies related to sediment and vegetation maintenance activities and mitigations.

- The Project preferred alternative would not comply with the *San Francisco Bay Water Quality Control Plan* (Basin Plan) requirement that impacts to wetlands and other waters of the State be avoided and minimized to the extent practicable.

- Mitigation for impacts on waters of the U.S. and waters of the State does not comply with the State and Regional Water Board policies.

COMMENTS

1) **Pre-selected Alternative.** The District only analyzed alternatives that were previously screened by the Corps for the Corp' Final GRR/EIS (March 2014). Therefore, the DEIR's alternatives analysis does not constitute a full array of feasible alternatives, so does not fully meet the CEQA requirements. This is particularly relevant because the Water Board cannot permit or certify the Project unless we concur with the lead agency's CEQA determination. As currently proposed, the Project does not meet the Water Board's policies, nor does it adequately meet CEQA requirements for reasons discussed in the following comments.

2) **Sediment Transport.** The Project will result in a wider and deeper channel than the existing channel morphology, but the DEIR does not explain how sediment will be transported through the Project reach. Without explaining sediment transport in the Project, the DEIR does not adequately describe the potential post-Project impacts or mitigations necessary to address impacts for sediment removal maintenance activities. The DEIR, section 3.1 (last paragraph) states:

   > *Because the proposed project is being designed to result in less erosion due to lower flow velocities, more stable bank design, and enhanced flow conveyance through bridges and culvert openings, operations and SMP2 maintenance actions associated with sediment removal and repair of eroded banks or access roads are likely to be reduced in magnitude compared to existing channel operations and maintenance activities.*

This statement is unfounded because the DEIR does not include data about existing sediment maintenance and how the Project will cause less sediment maintenance needs. In addition, without a sediment transport analysis, there is no evidence to show that the source of sediment is from eroding banks within the Project reach.

Water Board staff's best professional judgment regarding sediment transport in the Project reach is that the existing channel expresses a sustainable shape throughout the system, and the Project documents do not support that the proposed channel design is sustainable (Attachment A1 through A3). For example, the channel models could not identify depositional areas due to the ongoing maintenance to remove sediment (Attachment A-3: GRR/EIS, Appendix B, Part III-*Geomorpologic and Sediment Transport Assessment*, pg. 2-17). The existing channel width is consistently about 10 to 12 feet, including areas upstream and downstream of the Project reach as Water Board staff observed on September 4, 2015 and as shown in the Corps' draft 60 percent 60 percent design plans (June 2015). The sediment processes in the Project reach will result in sediment accumulation and eventually the same channel dimensions as existing conditions. This could adversely impact flow conveyance, which would not be consistent with the Project objectives.

Based on these findings, the Project will require ongoing, repetitive maintenance for sediment removal, which will result in repetitive impacts on the creek habitat which the DEIR does not disclose. Although the DEIR states that the District plans to conduct sediment maintenance to maintain conveyance (sections ES-5, 3.5.2.1), the maintenance needs may exceed the District's Stream Maintenance Program ("SMP2") thresholds, but this is not addressed in the DEIR. Please revise the DEIR to adequately explain the sediment transport processes in the Project, and the associated impacts due to future sediment maintenance activities and mitigations for the impacts.

3) **Project Objectives.** The DEIR lists the following three objectives for the Project (section 2.3.5):

- *Objective 1:* Reduce flood damages from Berryessa Creek upstream of Calaveras Boulevard throughout the study reach during the 50-year period of analysis beginning in 2017. Completed project would meet FEMA certification standards in all 4 project reaches.
- *Objective 2:* Use environmentally sustainable design practices in addressing the flood risk management purpose of the project wherever possible within the study reach, including taking advantage of restoration opportunities that may be pursued incidentally to the flood damage reduction purpose.

- *Objective 3:* Be consistent with Berryessa Creek Flood Risk Management Project Plan selected by USACE in the Director's Report of May 29, 2014.

Regarding Objective 2, the DEIR does not define "environmentally sustainable design practices." Please revise the DEIR to include the District's definition for this and to specify how the proposed Project meets this objective. Given Water Board staff's concerns regarding sediment transport in the Project (see Comment 2), the ongoing maintenance we anticipate will be necessary would not be consistent with an environmentally sustainable design.

Regarding Objective 3, the DEIR is not entirely consistent with the GRR/EIS because it does not include the GRR/EIS objective to "reduce sedimentation and maintenance requirements" (GRR/EIS, section 1.1). Please revise the DEIR to reconcile this discrepancy in consistency with the GRR/EIS.

4) **Impacts on Biological Resources.** The DEIR, section 2.5.5 states that the District plans to operate the Project under the District's existing Stream Maintenance Program (SMP2) for sediment removal tasks to maintain flow conveyance capacity and vegetation removal to maintain access and for fire prevention.

However, this contradicts the District's statement that the existing open water/aquatic vegetation (1.25 acres) and transitional vegetation ranging from the active channel to the channel uplands (up to about 3.27 acres) that will be removed for the Project would recolonize and thus serve to mitigate for what the District is calling a temporary impact that is less than significant with mitigation. The following excerpt is the District's rationale for this finding (section 3.5.5.1):

> *It is anticipated that wetland and transitional vegetation would regenerate naturally over the course of the first two growing seasons, and since the bottom width of the stream channel would be wider than under existing conditions, additional areas of wetland plant communities are likely to form. Because wetland vegetation would regrow after construction is complete and the area of wetlands vegetation would increase when compared to the existing condition, this impact would be less than significant.*

Water Board staff does not agree that the impacts would be less than significant, given that the DEIR contains no plans or evidence to support that the same or comparable hydrophytic vegetation would colonize naturally and meet or surpass the functions and values of the existing vegetation. In addition, the District plans to remove sediment and vegetation (section 2.5.5), so the assumption that the impacted vegetation would recolonize is unfounded.

Please revise the DEIR to include appropriate mitigation to compensate for both temporal and spatial losses in functions and values of the open water/aquatic vegetation and transitional vegetation. Such a plan would need to include, at least at the conceptual level, the types, numbers, densities, and locations of vegetation plantings, and success criteria. The details would need to be further developed in a mitigation and monitoring plan. We note that while the DEIR includes plans to hydroseed the banks to promote bank stabilization, particularly after coconut-fiber blanket biodegrade (3+ years), the DEIR does not discuss the nature of hydroseed (e.g., the species make-up), monitoring plans, or other details to demonstrate appropriate level of compensation for impacts on open water/aquatic and transition vegetation.

5) **Impacts on Beneficial Uses.** The DEIR repeatedly states or implies that the existing habitat is of marginal quality (e.g., sections 3.5.2.1, 3.5.2.3, and Table 3.12) and uses this as a basis for maintaining the status quo or even reducing the Project reach's beneficial uses. Water Board staff observed flowing and ponded water and egrets and mallard ducks in multiple sites along Reaches 1-3 during a site visit on September 4, 2015, despite the inspection occurring in the end of the dry season in the midst of a severe drought. These observations are consistent with the REC-2 (non-contact recreation such as bird-watching) and WILD (wildlife habitat) beneficial uses of the Project reach designated by the Water Board and listed in the Basin Plan, Table 2.1. The other beneficial uses are for body-contact recreation (REC-1); and warm water aquatic habitat (WARM). Because the Project would impact aquatic and transitional vegetation, the habitat the vegetation supports would be impacted. However, the DEIR does not address this. Please revise the DEIR to recognize the Project reach's designated beneficial uses and a plan to appropriately mitigate any unavoidable impacts on the creek habitat, especially the REC-2 and WILD beneficial uses.

6) **Description of Impacts on Creek Hydrology.** The District's alternatives analysis does not adequately address the potential of exposing the water table in new areas and resultant alterations in the creek's hydrology. Consequently, the DEIR does not include any mitigation for this potential impact on the post-Project hydrology. The Project would excavate to variable depths of 9 to 20 feet (Table 5.4). Given that the depth to groundwater ranges from about 7 to 20 feet below grade (DEIR, Appendix D-Geotechnical Report), the post-Project conditions would likely result in more area of the channel invert being in the groundwater table than existing conditions. Please revise the DEIR to address the post-Project hydrology conditions, and the impacts from vegetation and sediment maintenance activities on the creek's functions, values, and beneficial uses.

7) **Bank stabilization**
   A. **Discrepancies in DEIR and Appendix D.** The DEIR main body discusses that biodegradable coconut mats will be used for erosion control and bank stabilization (sections ES4, 2.5, and others). However, Appendix D-

Geotechnical Report (April 2015), section 2.1 states: “The erosion protection will consist of rip rap on the lower portion of the slope and geocells filled with aggregate or concrete on the upper portion of the slope,” and this is reiterated in section 23. In addition, Appendix D, section 12 states: “Rip rap is also being used for the channel invert between approximately Stations 115+00 and 164+00.” Please revise the DEIR to reference any inaccuracies and inconsistencies in the Geotechnical Report (or any other appendices, as appropriate). Please note that the Water Board staff has communicated to the Corps-District design team that the use of geocell bank stabilization does not comply with Water Board policies or the requirements in the Basin Plan to avoid and minimize impacts to the extent practicable.

B. **Hydroseed**. The DEIR states: “Channel banks would be protected with biodegradable erosion control blankets and hydroseeded” (ES-4; Table ES-2; section 2.5.2; and others). We caution that erosion control treatments such as hydroseeding, hydraulic mulch, tackifiers, soil binders, and straw mulch could wash into the channel rendering the erosion prevention method ineffective. Other soil bioengineering methods such as the planting of willow stakes and emergent in-stream vegetation could be used to stabilize the bed and banks below the mean high water level. Has the District considered integrating willow stakes or other bioengineering methods in the Project for bank stabilization?

8) **Alternatives Analysis for the 401 Certification**. Please note that for the Water Board to permit the proposed Project pursuant to the Clean Water Act, section 401, we require a project proponent to conduct an alternatives analysis consistent with the U.S. Environmental Protection Agency’s 404(b)(1) Guidelines. The Basin Plan incorporates the 404(b)(1) Guidelines by reference to determine the circumstances under which filling of wetlands, streams or other waters of the U.S. and/or the State, as the District proposes with this Project, may be permitted. In accordance with the Basin Plan, filling, dredging, excavating and discharging into a wetland or water of the state is prohibited unless the project meets the least environmentally damaging practicable alternative (LEDPA) standard as determined through the 404(b)(1) alternatives analysis. Although the LEDPA analysis is not required by CEQA, a project proponent may tailor their alternative analysis to fulfill both the CEQA and 404(b)(1) requirements to help expedite the Water Board’s Project review to issue a 401 Certification.

For example, during pre-CEQA interagency meetings, Water Board staff made suggestions that would help the Project meet the LEDPA standard by minimizing impacts in the creek and maximizing its beneficial uses (Interagency meetings, August 4 and August 11, 2015). This input includes: (1) planting willow stakes in the streambed edges; (2) installing the proposed pre-cast concrete culverts at grades that allow the formation of earthen bottoms; (3) using bioengineering methods in place of concrete for bank armoring and/or some or all floodwalls; and (4) identifying opportunities to maximize both flood conveyance capacity and opportunities for future adaptive management of the channel by increasing channel cross section. For example, such adaptive management practices could be completed where the Corps' preferred alternatives propose reaches with maintenance access roads on both sides of the channel, by removing or lowering the road on the non-multi-purpose path side.

The District did not incorporate the Water Board staff's suggestions in the CEQA analysis, except for DEIR Alternative 4. At three times the cost of the District-preferred alternative, Alternative 4 is cost-prohibitive because it apparently incorporates the "all options" scenario (though this is not explicitly explained in the DEIR). Water Board staff recommends the District revise the CEQA alternatives analysis to include feasible alternatives to meet the LEDPA standard. This would help expedite Water Board staff's Project review for the 401 Certification process.

In summary, Water Board staff appreciates the opportunity to provide comments on the DEIR. The DEIR is well-organized, but it does not adequately describe the proposed Project's environmental impacts and associated mitigations. In addition, the proposed Project would not meet the Water Board's requirements for project proponents to avoid and minimize impacts and to appropriately compensate for any unavoidable impacts in accordance with the Basin Plan and (404(b)(1) Guidelines. If you have any questions about our comments, please contact Susan Glendening of my staff at (510) 622-2462 or via email to Susan.Glendening@waterboards.ca.gov.

Sincerely,

William B. Hurley
Senior Engineer

Attachments:

A-1: Section 6.2 excerpt from the *GRR/EIS,* March 2014

A-2: Pages iii, and A-4 through A-6 from the *Final Independent Peer Review Report, Berryessa Creek*, March 6, 2013

A-3: GRR/EIS, Appendix B, Part III-*Geomorphologic and Sediment Transport Assessment,* March 2012

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Attachment A-1

GRR/EIS, Section 6.2

General Reauthorization Report/Environmental Impact Statement

Prepared by U.S. Army Corps of Engineers, Sacramento

March 2014

FINAL EIS 2015

6.2 COMPARISON OF ALTERNATIVE PLANS

The purpose of this step is to compare the results from the evaluations completed, for the purpose of developing a recommended plan that addresses the flooding problems in Berryessa Creek. A more detailed project footprint, including temporary construction easements, staging areas, and access routes, is presented in the overview exhibits at the end of Chapter 6.

6.2.1 Hydraulic Design

6.2.1.1 Hydrologic Effects

With-project discharges are actually higher within the creek than the without-project discharges. This is typical of flood risk management projects that maintain flow within the channel that otherwise would overflow onto the floodplain in the without-project condition. The discharges for the without- and with-project conditions upstream of I-680 remain the same in Alternatives 2A/d and 4. On the other hand, the difference between without- and with-project discharges upstream of I-680 is less pronounced in Alternative 5.

6.2.1.2 Water Surface Profiles

The with-project water surface elevations resulting from the additional discharge in Alternatives 2B/d, 4/d, and 5 are generally higher than in Alternative 2A/d, but the amount of increase is highly variable. These results are for fully contained flows. Comparison to existing conditions is therefore hypothetical only; the computed without-project (Alternative 1) water surface elevation at any point assumes full containment at each upstream section, and flows are restricted to the extent of each cross section in the event of breakout.

Among different alternatives, the different channel configurations downstream of I-680 affect water surfaces that vary by reach. The vegetated terraces in Alternative 4/d tend to reduce the available conveyance in the channel in comparison to Alternatives 2A/d and 2B/d.

6.2.2 Sediment Transport

The quantitative sediment analysis was conducted for the without-project, Alternatives 2A/d, 2B/d, and 4/d using hydraulic models developed for previous phases of this study for existing conditions between Old Piedmont Road and I-680. In addition, analyses were conducted for Alternatives 2B/d and 4/d assuming the proposed SCVWD bypass alternative was in place between Old Piedmont Road and I-680.

The analysis indicated an increase in sediment transport through the I-680 to Montague Expressway and Montague to Calaveras Boulevard for Alternatives 2A/d and 2B/d. The increased transport results in a decrease in deposition in the I-680 to Montague reach for the alternatives. With a larger amount of sediment being transported through the upstream reach, there is an increase in the amount of deposition in the Montague to Calaveras Boulevard reach for all alternatives over the without-project alternative. Overall, the total amount of sediment deposited in the study area for Alternatives 2A/d and 2B/d is nearly equal to that under the without-project conditions. In contrast, the analysis showed a marked increase in deposition in for Alternative 4/d.

upstream

The analysis also showed a significant reduction in the deposition in the sediment basin below the Piedmont-Cropley culvert over existing conditions. This is due to a majority of flood flows being transported through the proposed SCVWD bypass culvert. The reduction in the flood flows to the Greenbelt reach results in a significant reduction in the sediment supply to the downstream reach. The sediment supply conveyed through the bypass culvert adds to the supply to the downstream reach, but accounts for only a small portion of the reduced Greenbelt sediment supply. The sediment transport rate for the Morrill to I-680 reach is greater than the combined sediment supply for the Greenbelt and bypass culvert. Since the sediment transport capacity through the reach is greater than the incoming supply, no deposition is seen in the reach. For Alternatives 2B/d and 4/d, there is an increase in sediment transport through the I-680 to Montague and Montague to Calaveras reaches over the without-project alternative. The increased transport results in no deposition in the I-680 to Montague reach. Normally, a larger amount of sediment being transported through the upstream reach would result in an increase in the amount of deposition in the Montague to Calaveras Boulevard reach. But since the supply from the Greenbelt reach is limited, the transport capacity of Alternative 2B/d can transport the entire supply to the downstream reach with no deposition and Alternative 4/d showing a small amount of deposition.

Throughout the study area, there are large variations in velocities and shear stresses that can cause localized sedimentation and scour problems. During the design phase, the project design needs to be further refined to reduce the level of these changes. Additionally, the measures used to provide passage of the design event through bridges should be reviewed. There may be the creation of significant backwater conditions in cases in which walls were extended above the bridge deck to contain flows. The reduced velocity and shear stress may cause an additional potential for additional, localized deposition in an area that in some cases already experiences deposition.

Currently, the study area is a deposition zone, and a reduction in velocity will further increase deposition and the need for maintenance. Constructed features should facilitate removal of deposited sediments.

6.2.3 Floodplains

The final array of alternative plans was analyzed using the Lower Berryessa Creek FLO-2D model. Of the four project alternatives, only Alternatives 2A/d and 5 have breakouts from the Berryessa Creek channel for the modeled events. Alternatives 2B/d and 4/d were developed to meet FEMA certification requirements using risk-based principles assuming SCVWD's bypass structure upstream of I-680 is implemented. The bypass design resulted in higher flow rates at I-680 resulting in Alternatives 2B/d and 4/d to have a larger conveyance capacity allowing both alternatives to convey up to the 0.002 exceedance probability event. Thus, no residual floodplains were mapped for these alternatives.

Attachment A-2

Pages iii; A-4, A-5, and A-6

Final Independent External Peer Review Report
Berryessa Creek, Santa Clara County, California,
General Reevaluation Study (GRS)

March 6, 2013

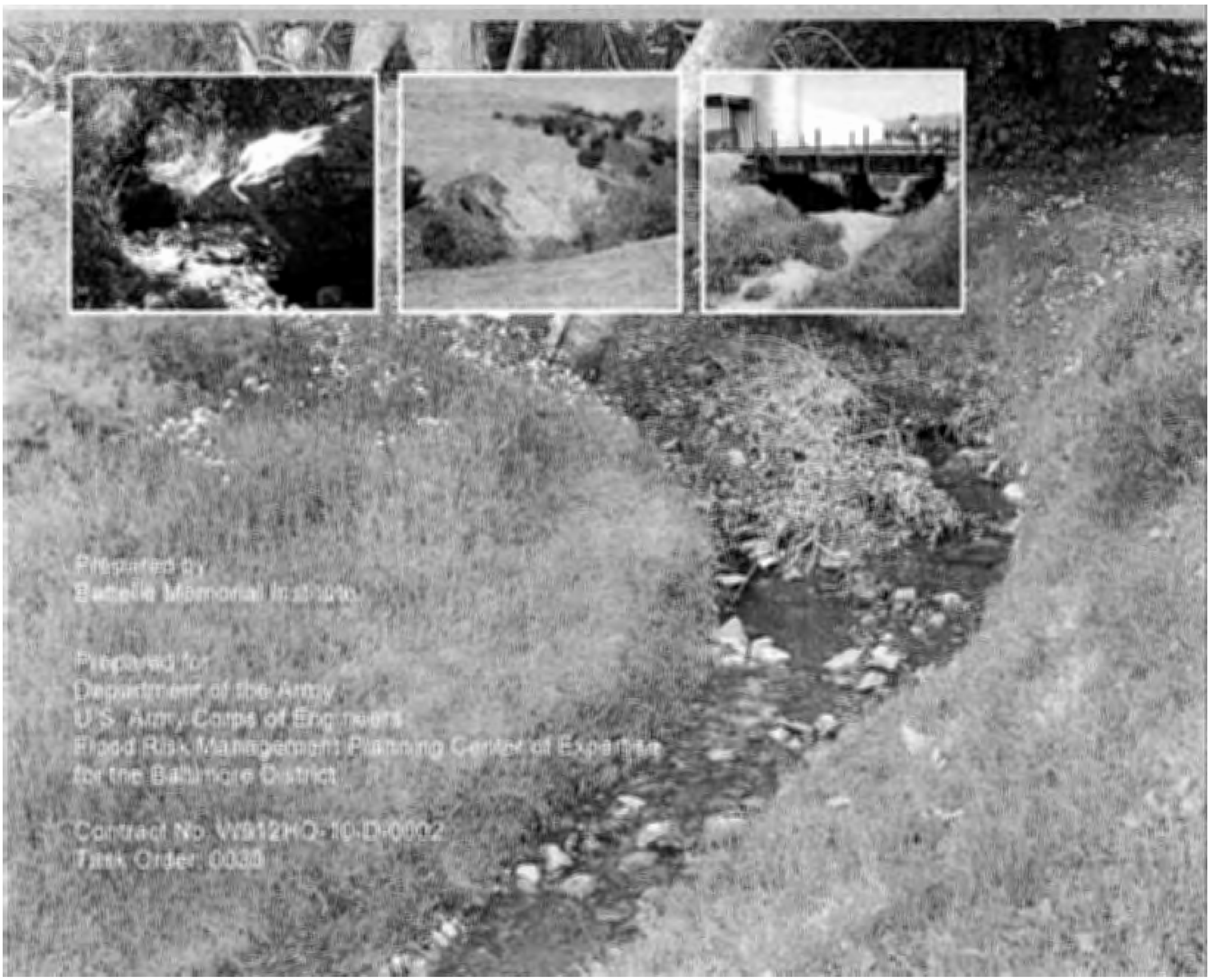
Prepared by Battelle Memorial Institute

Prepared for Department of the Army
U.S. Army Corps of Engineers
Flood Risk Management Planning Center of Expertise for the Baltimore District
Contract No. W912HQ-10-D-0002
Task Order: 0030

Battelle
The Business of Innovation

March 6, 2013

Final Independent External Peer Review Report Berryessa Creek, Santa Clara County, California, General Reevaluation Study (GRS) Draft General Reevaluation Report and Environmental Impact Statement/Environmental Impact Report



Prepared by
Battelle Memorial Institute

Prepared for
Department of the Army
U.S. Army Corps of Engineers
Flood Risk Management Planning Center of Expertise
for the Baltimore District

Contract No. W912HQ-10-D-0002
Task Order: 0038

these, six were identified as having high significance, eight had medium significance, and one had low significance.

Results of the Independent External Peer Review

The panel members agreed among one another on their "assessment of the adequacy and acceptability of the economic, engineering, and environmental methods, models, and analyses used" (USACE, 2012; p. D-4) in the Berryessa Creek review documents. The Panel found that, overall, the Berryessa Creek report is well organized and comprehensive. An extensive array of engineering measures was considered in the development of alternatives and the criteria to eliminate plans from future study are well described and logical although the impact of sedimentation on the channel design has not been considered adequately. Table ES-1 lists the Final Panel Comment statements by level of significance. The full text of the Final Panel Comments is presented in Appendix A of this report. The following statements summarize the Panel's findings.

Engineering – The Berryessa Creek GRS/Draft GRR/EIS/EIR contains extensive details on the hydrologic and hydraulic analyses performed. In general, the assumptions that underlie the engineering aspects are technically sound and appropriate. The hydrologic and hydraulic modeling procedures as presented in the report are technically sound and acceptable. Although the report presents overwhelming evidence of sedimentation issues within the project area, neither the impact of sedimentation issues on the channel design nor details on the maintenance activities with relation to sedimentation have been presented. In addition, there are insufficient details on the maintenance activities with relation to sedimentation. The Panel has expressed significant concern about the lack of details on the operation and maintenance (O&M) plan and has identified the need for a detailed O&M plan to ensure the design assumptions concerning sedimentation are valid.

Economics – The Panel determined that the adequacy and acceptability of the structure and content values, total annual costs, and the results of the economic risk analysis could not be determined due to lack of documentation. The report does not describe the methods used to develop the structure inventory, conduct and verify the content survey, and calculate structure values. The Panel was unable to determine if the structure and content data used in the analysis are accurate and if they reflect the current conditions in the study area. Several issues pertaining to the calculation of annual equivalent damages (AED) to structure and content, the unexplained increase in benefits resulting from the incorporation of risk and uncertainty, and the presentation of the results of the economic analysis are identified that could significantly impact the findings of the economic analysis. In addition, the report contains little documentation describing the development of the lands, easements, rights-of-way, relocations, and disposal areas (LERRD) costs and the annual operation, maintenance, repair, replacement, and rehabilitation (OMRR&R) costs, preventing an accurate assessment of the total annual costs used in estimating the benefit to cost ratios. Based on the analysis presented in the reviewed documents, the Panel cannot accurately assess the economic feasibility of the Recommended Plan.

Environmental – The Berryessa Creek GRS/Draft GRR/EIS/EIR adequately describes existing conditions of vegetation in the project area, but does not include a thorough review of special-

Final Panel Comment 1

The impact of sedimentation is not included in the hydraulic modeling aspect of channel design.

Basis for Comment:

The Main Report and Appendices provide overwhelming evidence of active sediment transport throughout the project reach as explained below:

- Appendix B, Part III, Section 2.2.1 describes the presence of a high sediment production zone in the upper watershed with erosive soils/landslides and steep channels capable of transporting the large quantities of sediment to the downstream watershed.
- Appendix B, Part III, Section 2.2.1.4 (p. 2-17) states that HEC-6T sediment modeling results indicate "a mixture of aggradation and degradation scattered throughout the project area."
- Main Report, Section 2.2.1.1 presents the results of sediment yield analysis showing estimated sediment delivery as:
  1. Berryessa Creek at Old Piedmont Road = 9,900 tons/year
  2. Sweigert, Crosley, and Sierra Creeks = 1,900 tons/year
  3. Piedmont Creek = 700 tons/year
  4. Arroyo de los Coches = 3,200 tons/year.
- Appendix B, Part III, Section 2.2.2 presents the sediment removal history based on Santa Clara Valley Water District (SCVWD) maintenance records. These records show sediment removal occurring throughout the project area.
- Appendix B, Part III, Section 2.2.2 (p. 2-21) describes the possibility of sediment being transported through the project area to the reach downstream of Calaveras Boulevard.
- Main Report, Section 2.4.1 states, "Winter flows tend to be turbid, due to sediment loading from the surrounding foothills and from bank erosion along the creek."
- Appendix B, Part I, Section 5.3.2 states, "Based on the observations of David Adams of the SCVWD, sediment removed in the maintenance reaches upstream of Calaveras Boulevard is approximately uniformly distributed within each channel reach (rather than concentrated at bridge locations)."

Although there is overwhelming evidence that sedimentation occurs throughout the project reach, according to Main Report, Section 4.4.2.6, "For the hydraulic analysis, it was assumed that the channel is in its maintained state with the sedimentation basin downstream of Piedmont-Cropley cleaned out and the invert of bridges the same as those in the USACE model."

The hydraulic modeling performed in the study assumed clear channel conditions and did not analyze the potential reduction in channel capacity due to sediment deposition in the channel bed. In addition, high sediment concentrations can create "bulking" (Mussetter et al., 1994) of the flows, where the sediment volume becomes significant compared to water volume so that higher water surface elevations may result due to the presence of suspended sediment load. The impact due to "bulking" of flows is not considered as part of the hydraulic (HEC-RAS and FLO-2D) modeling. The design discharges were not adjusted to accommodate "bulking" of the flows due to sediment load.

Significance – High:

Reduction in channel capacity due to sediment deposition and bulking can impact the flow containment and extent of flooding, which will affect the project objective of reducing flood damages and the level of risk reduction achieved can be less than the project objective of 90-95 assurance for the 1-percent flood event.

Recommendations for Resolution:

1. Investigate post-sedimentation within the channels using post-sedimentation cross-sections from the sediment transport model.
2. Adjust design discharges to accommodate bulking of the flows due to sediment load.

Literature Cited:

Mussetter, R. A., P.F. Lagasse and M. D. Harvey (1994). Sediment Erosion and Design Guide. Prepared for the Albuquerque Metropolitan Arroyo Flood Control Authority by Resource Consultants and Engineers, Inc., Fort Collins, CO.

Final Panel Comment 2

The operations and maintenance plan does not present sufficient details related to sediment removal and maintenance of clear channel conditions.

Basis for Comment:

Sediment management is key to the success of the project as the project design is developed on the assumption of clear channel conditions. It is critical to ensure that the operations and maintenance (O&M) plan contains adequate details describing the process that will be adopted to maintain the channel through sediment removal. However, the O&M plan as presented in the Main Report Section 7.4 consists of only a single paragraph and does not provide sufficient details on the sediment removal process, sediment removal locations, or sediment removal frequency.

There are other sections of the Main Report that discuss the need for sediment removal through maintenance:

- Main Report (p. 2-17) describes the significant blockage of the Cropley and Piedmont Culvert.
- Both the Authorized Plan and the National Economic Development (NED) Plan identified removal of sediment at the downstream face of I-680 as a project task.
- Appendix B, Part III, Section 3.1.1 describes the need for sediment removal maintenance to preserve adequate flood conveyance capacity.
- Appendix B, Part III, Section 3.1.4 describes the need for identifying and creating designated locations for sedimentation-related maintenance activities.
- Appendix B, Part III, 3.1.5.2 describes the need to maintain vegetation growth within the channels so that sediment can effectively be conveyed by the channel.

In addition, the hydraulic analysis presented in Main Report, Section 4.4.2.6 assumes clear channel conditions without sediment depositions in the channel bed. The Authorized Plan had identified a primary sediment basin near Old Piedmont. In comparison, the NED Plan does not include any improvements upstream of I-680 and therefore does not include a sediment basin to capture the sediment from the upper watershed. As a result, sediment deposition can occur at various locations within the project study area. This section of the report, as well as the Section 7.4 on operations and maintenance, does not clearly describe how the sediment maintenance will be performed or identify all the locations where sediment removal will be performed.

One of the statements presented in Appendix B, Part III explains that existing deposition trends will be exacerbated due to design modifications. The with-project conditions are expected to worsen the sediment deposition, so additional maintenance efforts may be required to counter the increased sedimentation. No details on additional maintenance requirements are presented in this appendix.

Appendix B, Part III (p. 2-21) discusses the possibility of increased deposition in the reach below Calaveras Boulevard. The main report does not present any discussion on downstream impacts and mitigation needed to reduce the amount of sediment carried to downstream reaches outside the project study area.

Attachment A-3

Appendix B, Part III: Geomorphologic and Sediment Transport Assessment

General Reauthorization Report and Environmental Impact Statement

Berryessa Creek Element

Coyote and Berryessa Creeks, California

Flood Control Project

Santa Clara County, California

March 2012

Berryessa Creek Element
Coyote and Berryessa Creeks
Flood Control Project
Santa Clara County, California

Appendix B: Engineering and Design

Part III

Geomorphologic and Sediment Transport Assessment

BERRYESSA CREEK PROJECT

APPENDIX B, Part III: Geomorphic and Sediment Transport Assessment

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CHAPTER 1: INTRODUCTION

This appendix is Part III of the engineering appendices supporting the Berryessa Creek Flood Control Project Post-Authorization Study. The engineering appendices are as follows:

- Part I. Hydraulic Analysis of Alternatives
- Part II. Floodplain Development
- Part III. Geomorphic and Sediment Transport Assessment
- Part IV. Design and Cost of Alternatives

This appendix refers to figures, tables, and results in the accompanying appendices and in the main body of the report. This appendix provides supporting fluvial geomorphology and sediment transport analyses for the formulation and evaluation of the Berryessa Creek Project Alternatives. A summary and interpretation of previous work related to the geomorphology of the system is also included. In addition, insight from observations by the project team is provided, particularly in reference to supply of sediment from the upstream watershed.

Sediment transport analyses of the existing condition are summarized in light of available sediment removal records. The results of the hydraulic analysis of the alternatives is utilized to qualitatively address potential changes in sediment transport conditions under project scenarios compared to the without-project condition. This information is utilized to provide recommendations on design refinements to address fluvial geomorphic and sediment transport aspects of the project design as well as recommendations for additional analyses to support the design effort.

Figure 1-1 shows the delineations of watersheds draining to the project area, as presented in the NHC hydrology report (2003). Figure 1-2 shows the project footprint relative to the road crossings and other features within the project area.

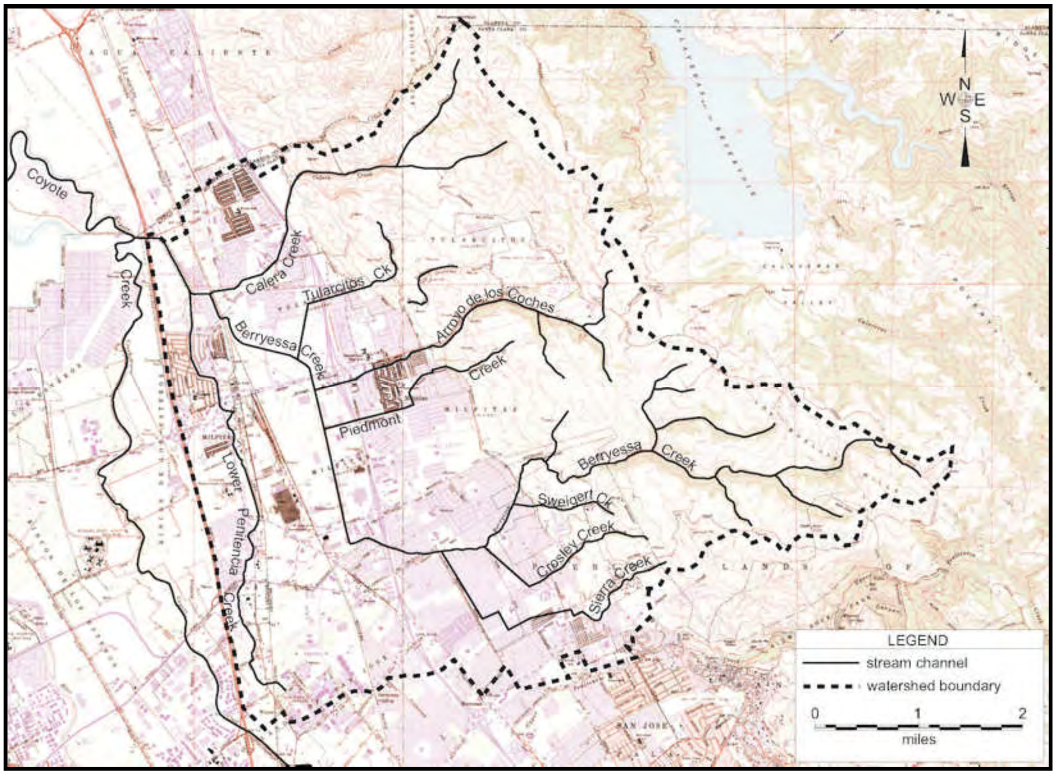

Figure 1-1 Watershed Map (Source: NHC 2003)

Figure 1-2 Project Footprint

A number of issues were identified as important for this analysis to address. An evaluation of the stability of the alternatives in terms of their sediment transport response is necessary. Because of the urbanized nature of the area and the limited area available for the project, it was determined early in the plan formulation process that the channel would be protected in most areas to prevent erosion. However, the channel bed will remain mobile so it is necessary to assess the potential for channel bed aggradation and degradation. The project alternatives should be designed to prevent excessive scour or deposition. The influence of the proposed alternatives on sediment removal requirements is another important issue. Historically, sediment removal in the project area (see Table 2-1) has averaged on the order of 1,046 cubic yards per year upstream and 616 cubic yards per year downstream of I-680 for the project reach with a total of 7,179 cubic yards per year from the entire Berryessa Creek channel. Also tied to sediment removal is the potential for changes to the existing sediment retention basin and construction of additional sediment management structures under consideration by others. The Corps GDM (USACE 1993) included a sediment basin above Old Piedmont Road. To address issues surrounding the reconfiguration of the sediment basin, the watershed was evaluated to determine if there were areas further upstream in which sediment management activities could be applied to reduce sediment delivery to the basin area.

Besides the sediment transport aspects of the design, fluvial geomorphology concepts were applied to evaluate the design and provide recommendations for potential refinements as necessary. Though the project is located in a highly urban environment with limited right of way and numerous constraints created by bridges, roads, utilities, and buildings; the concepts of fluvial geomorphology are still useful in developing an appropriate design. These concepts can help in evaluating the system response to the alternatives and provide input on ways of developing a more sustainable project in terms of maintenance and environmental quality. Application of fluvial geomorphology assisted in the evaluation of the sediment transport issues identified in the previous paragraph. In addition, recommendations for sizing the channel and evaluation of the response of the Greenbelt Reach, which will not be as constrained as the project area, are addressed.

The with-project alternatives evaluated in the current effort were carried forward from the conceptual alternatives presented in the F3 report (Tetra Tech 2004) and subsequently narrowed down to three alternatives by the Corps. Typical cross sections of each alternative are presented in *Appendix B, Part IV: Design and Cost of Alternatives* in this engineering appendix. An important purpose of these alternatives was to evaluate large-scale economic issues between general approaches to flood control. Alternative 1 is the without-project condition. Project alternatives under consideration by others include floodwall construction and excavation of a floodplain terrace within the Greenbelt Reach upstream of I-680 along with a high-flow bypass culvert running beneath Cropley Road. Downstream of I-680, Alternatives 2A/d and 2B/d were formulated to provide flood control utilizing channel excavation and bridge modifications to increase conveyance in a project footprint that could be constructed within the existing right of way. As a result, a large main channel is excavated that has the capacity to convey the 1% chance exceedance event. Alternative 2A/d is designed to pass the 1% chance exceedance event with a 50% conditional non-exceedance probability (CNP) using risk and uncertainty principles with Alternative 2B/d passing the 1%

chance exceedance event with 90% CNP (meeting the FEMA certification criteria). Levees or floodwalls are extended as needed to maintain a consistent capacity throughout the project with the appropriate certainty. Alternative 4/d incorporates vegetated floodplain benches along the low-flow channel, with concrete floodwalls extended vertically from the outer edges of the floodplain bench. This allows Alternative 4d/ to be constructed within the existing right of way.

Alternatives 2B/d and 4/d include the complete replacement of all bridge and culvert crossings with the exception of the Ames Avenue and Yosemite Drive crossings, which would require shoring/stabilization of existing abutments and construction of transition structures, and the I-680 crossing, which would not be affected. Modifications within channel reaches include excavation and levee/floodwall construction. Levees, floodwalls, and tops of bank are designed according to risk and uncertainty principles. Further details on the flow profiles and modeling methodology are described in *Appendix B, Part I: Hydraulic Analysis of Alternatives* in this engineering appendix. The analyses and recommendations presented in this appendix will be utilized to guide future sediment transport modeling efforts supporting more detailed designs that are carried forward.

CHAPTER 2: EXISTING CONDITIONS

2.1 Summary of Geomorphology

This report generally assesses the impacts of the sediment generated in the upper watershed on the proposed project alternatives in the lower watershed. Two primary documents provide information describing the geomorphology of Berryessa Creek within the project area and the upstream watershed: the Sacramento District's GDM (USACE 1993) and "Upper Berryessa Creek GRR Basin Geomorphology Technical Memorandum" (NHC 2001). "An Urban Geomorphic Assessment of the Berryessa and Upper Penitencia Creek Watersheds in San Jose, California," a Colorado State University dissertation by Jordan (2009), contains data and conclusions applicable to the site geomorphology and will likely be published in the near future. Preliminary results and analysis methods are summarized at the end of this report in Addendum 1. In addition, Tetra Tech has conducted several site visits to the project area and the upstream watershed to observe and document conditions related to fluvial geomorphology. The summary of existing geomorphic conditions is based on these three sources.

2.1.1 Geology and Soils

The Berryessa watershed consists of two distinct landforms. The watershed above the urbanized area is mountainous terrain consisting of the Los Buellis Hills, part of the Diablo Range. The highest point in the watershed is Monument Peak at an elevation 2,594 feet. Within the project area, Berryessa Creek flows across an alluvial fan created by Berryessa Creek and its tributaries. The minimum elevation in the watershed is 3 feet at the confluence with Penitencia Creek. At the downstream limits, Berryessa Creek is tidally influenced. Under existing conditions, the upland portion of the watershed is mostly undeveloped with a few residences scattered mostly along the basin divide. The primary land use in the upland portion of the watershed is grazing. Due to zoning practices, the future condition is not anticipated to change significantly in terms of land use. In contrast the alluvial fan portion of the watershed is almost entirely urbanized.

In the uplands, the geology consists mainly of Tertiary and Quaternary age sedimentary rocks composed primarily of sandstone, siltstone and shale. Minor tuff, claystone and partially to completely serpentinized ultramafic rock outcrop in the basin in smaller amounts (NHC 2001). As shown in Figure 2-1, two major faults cross the lower and upper extents of the watershed. The Hayward Fault zone trends across the base of the Los Buellis Hills and the Calaveras Fault passes along the upper watershed boundary. These two major faults and numerous minor faults cross the Berryessa Creek watershed in northwest to southeast direction.

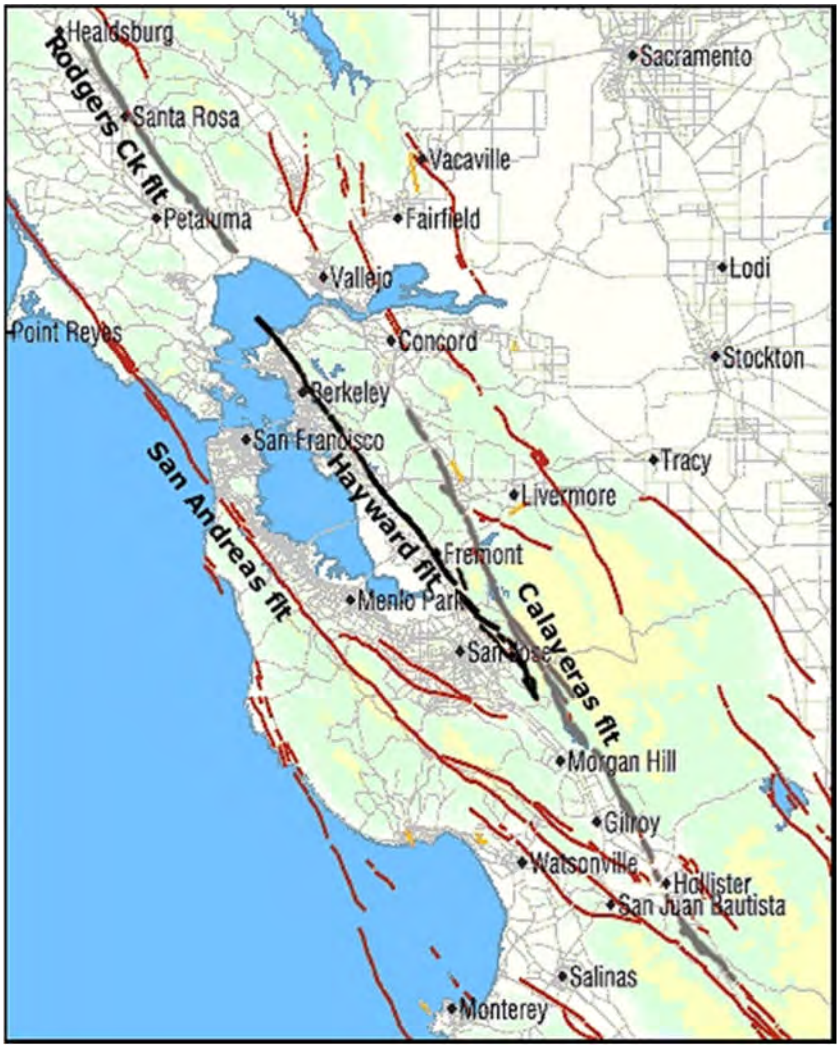

Figure 2-1 Bay Area Fault Zones (Source: USGS)

An important feature of the watershed occurs in the Hayward Fault zone, an area referred to in the previous reports as the "canyon" reach, extending from about 1,000 to 4,000 feet upstream of Old Piedmont Road. Underlying bedrock in this reach is composed of poorly consolidated, highly fractured Tertiary age rocks that contain swelling clays (NHC 2001). This is a high sediment production zone with erosive soils, large sediment supply from landslides, and a steep channel section capable of transporting large quantities of sediment. This is the only reach observed during the Tetra Tech watershed reconnaissance that had evidence of debris flows and transport of large boulders, several feet in diameter and larger. It also contained the only adjacent watershed area that was observed to have numerous active landslides scarps. The GDM (USACE 1993) supports this statement, indicating, "Upstream of the canyon zone, the ravines in Berryessa Creek and its larger tributaries are well treed and appear to be relatively stable."

Soils in the upland portion of Berryessa Creek are said to be of two types: clay loams on the relatively gentle slopes, and coarse rocky or gravelly soils on steeper slopes. Both types are derived from the underlying sedimentary rocks, the clay loams by weathering and vegetation, and the rocky soils by physical disintegration especially in the fault and shear zones (USACE 1993).

The geology of the alluvial fan in the Santa Clara Valley portion of the watershed is limited to Quaternary age, semi-consolidated alluvium near the base of the Los Buellis Hills with younger, unconsolidated alluvium further downslope. The alluvial sediments are largely fine grained, consisting primarily of moderate to poorly sorted fine sand, silt, and clay (NHC 2001). Borehole data from this lower portion of the creek, particularly downstream of I-680 show the creek to be underlain by large amounts of clayey soils.

In general, the Santa Clara Valley is underlain by some 1,000 to 1,500 feet of alternating estuarial and alluvial fan deposits of Quaternary age. The estuarial deposits were laid down under episodes of marine flooding and the alluvial fans during dryland episodes when the sea level was lowered during the major glaciations. The surficial materials in the valley are partly coarse alluvial fan deposits from stream channels, and partly fine materials derived from suspended load deposition during floods in areas between the stream channels (USACE 1993).

Within the project area, the streambanks are formed of fairly erosion-resistant material; the soils contain a large clay component primarily consisting of silty and sandy clay. Upstream of I-680, soils retain a significant clay component but exhibit more frequent clayey silt and clayey sand lenses with occasional gravels (NHC 2001). As a result, eroded sections of streambanks in this area are near vertical. Within the project area, bed material is somewhat variable due to the high level of channel alteration and the presence of numerous bridges and several other hydraulic structures. In general, the bed material is composed of sands and gravels. The average distribution for the entire urbanized reach upstream of Calaveras Boulevard, as presented in NHC (2003), is 28 percent sand, 69 percent gravel and 3 percent cobble with a median diameter of 5.5 mm (fine gravel).

The watershed upstream of Old Piedmont Ave. was broken into reaches with common characteristics based on field observations. Classification of these characteristics by reach allows for explanation of sediment transport-related trends and prediction of future erosion and deposition zones on a qualitative basis. The reach breakdown is shown in Figure 2.2 along with the locations of photographs presented below.

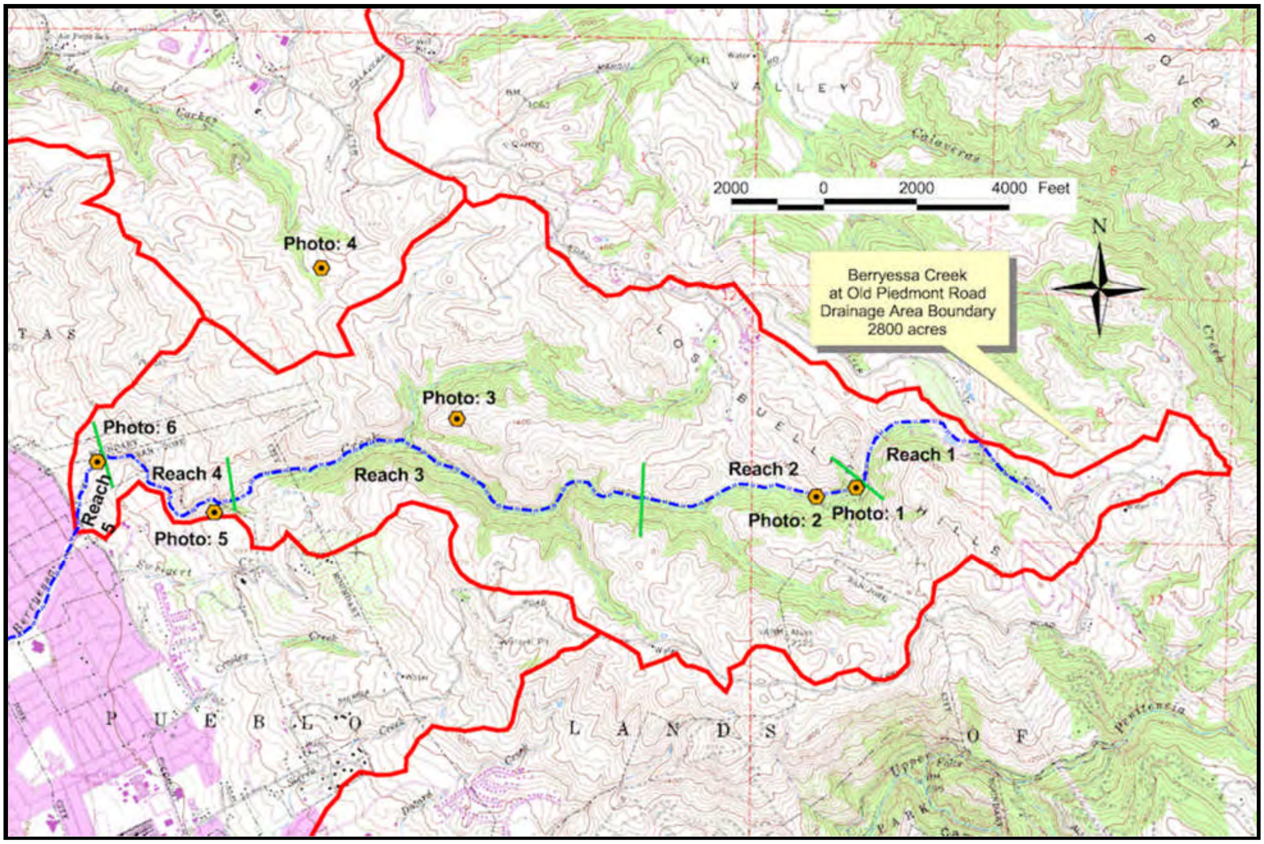

Figure 2-2 Upper Watershed Boundary, Reaches, and Photo Locations

2.1.2 Stream Profile

There is a distinct difference between the profile of Berryessa Creek in the uplands and on the alluvial fan within the Santa Clara Valley. Figure 2-3 shows the profile for the entire length from the estuary downstream from the confluence with Coyote Creek, upstream to the headwaters. Within the valley reach, which includes the project area, the channel gradient averages less than 1 percent. In contrast, the upland reach averages over 6 percent.

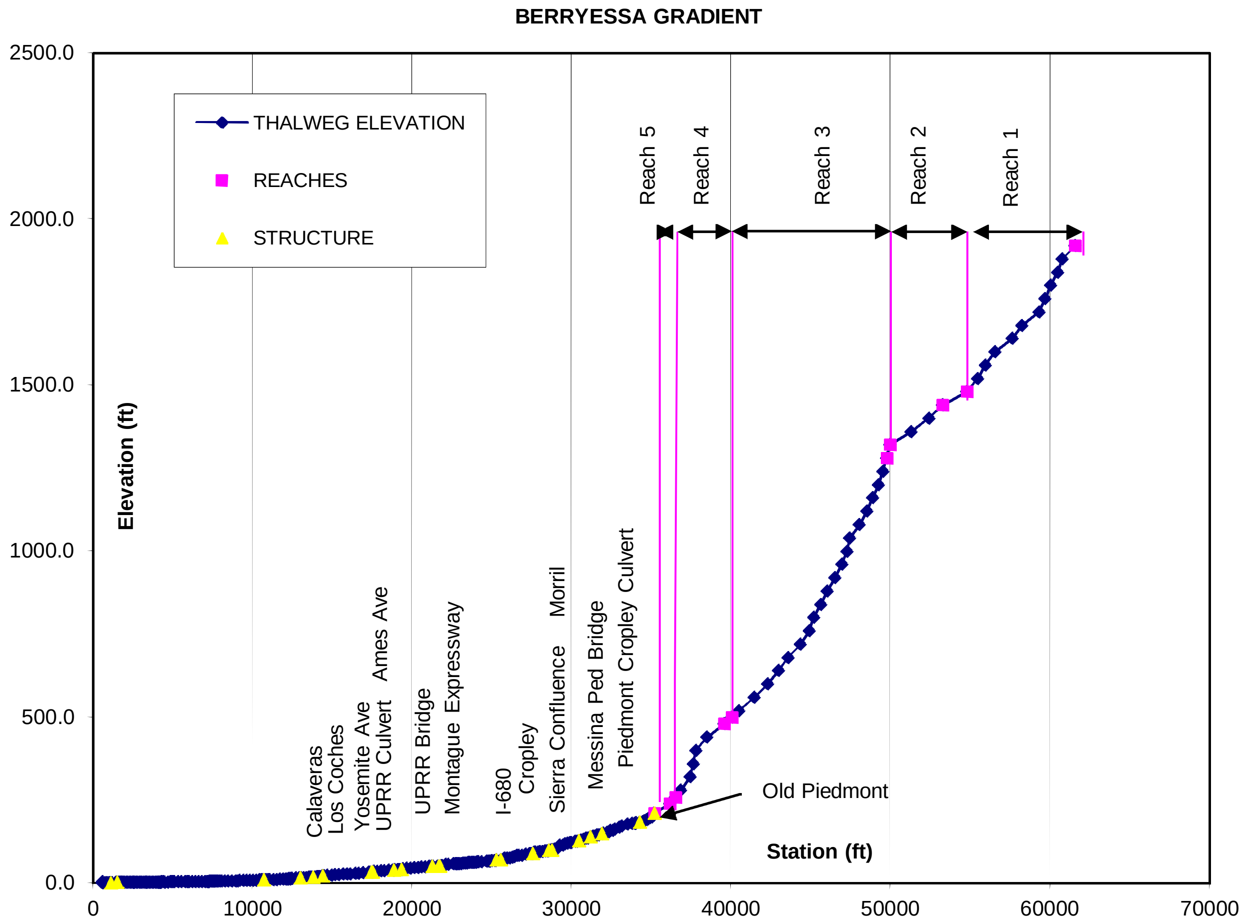

Figure 2-3 Berryessa Creek Profile from the Estuary to the Headwaters

Upstream of Calaveras Boulevard, the gradient follows the expected pattern of downstream reduction, with one exception. Starting at Old Piedmont Road, channel gradients are listed below:

Old Piedmont Road to Cropley Avenue	0.0271
Cropley Avenue to D/S of Piedmont Sediment Basin	0.0180
D/S of Sediment Basin to U/S of Sierra Cr. Drop	0.0156
Drop Structure to Cropley Avenue	0.0135
Cropley Avenue to I-680	0.0106
I-680 to Montague Expressway	0.0035
Montague Expressway to Calaveras Boulevard	0.0049

The channel leaves the uplands at a gradient of about 3 percent and gradually reduces to a slope on the order of 1 percent at I-680. However, below I-680, the gradient abruptly decreases by a factor of 3 to 0.35 percent between I-680 and Montague Expressway. Below Montague, the slope increases to approximately 0.5 percent.

There are numerous bed controls throughout the project area. These are formed by bridges or box culverts with concrete bottoms, drop structures, and segments of channels lined with concrete. Figure 2-4 identifies locations along the profile that act as grade controls.

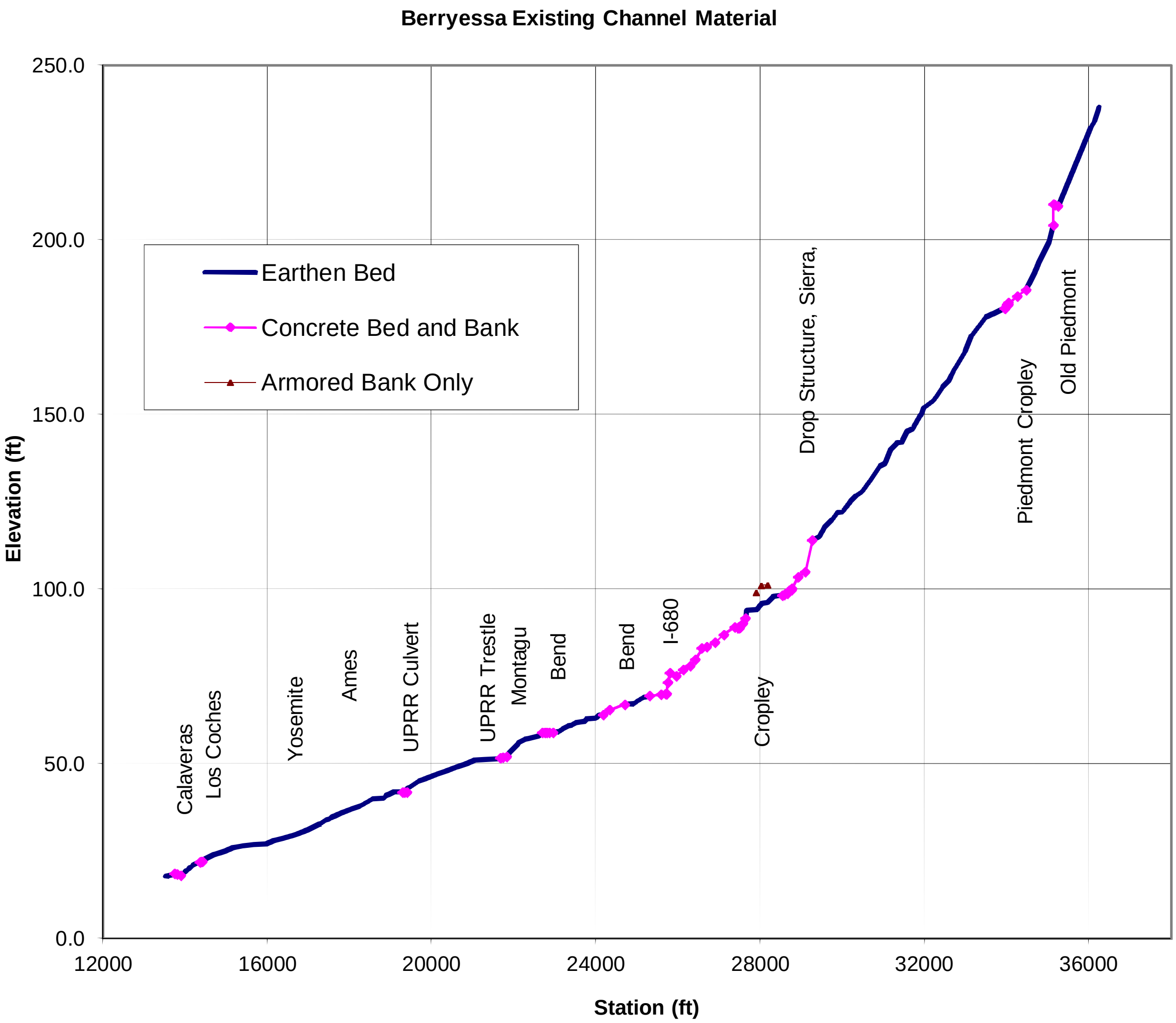

Figure 2-4 Location of Current Bed Controls along Berryessa Creek

The stream through the upper watershed was divided into five segments. Figure 2-5 provides a profile of the upland portion of Berryessa Creek. For the upper 1.3 miles, the gradient averages 6.5 percent. For about a mile, the gradient flattens to 3 percent. The gradient increases for the next two miles, averaging 8 percent with a gradual decrease in the downstream direction. The gradient then picks up as the stream crosses the Hayward Fault zone and passes through the "canyon" reach (Reach 4). The average gradient thought this segment is 8 percent with a portion of the stream near the center of the reach with a gradient of 15 percent. In the downstream 1,500 feet above Old Piedmont Road, Berryessa Creek transitions from the uplands to the alluvial fan with an average gradient of 4 percent.

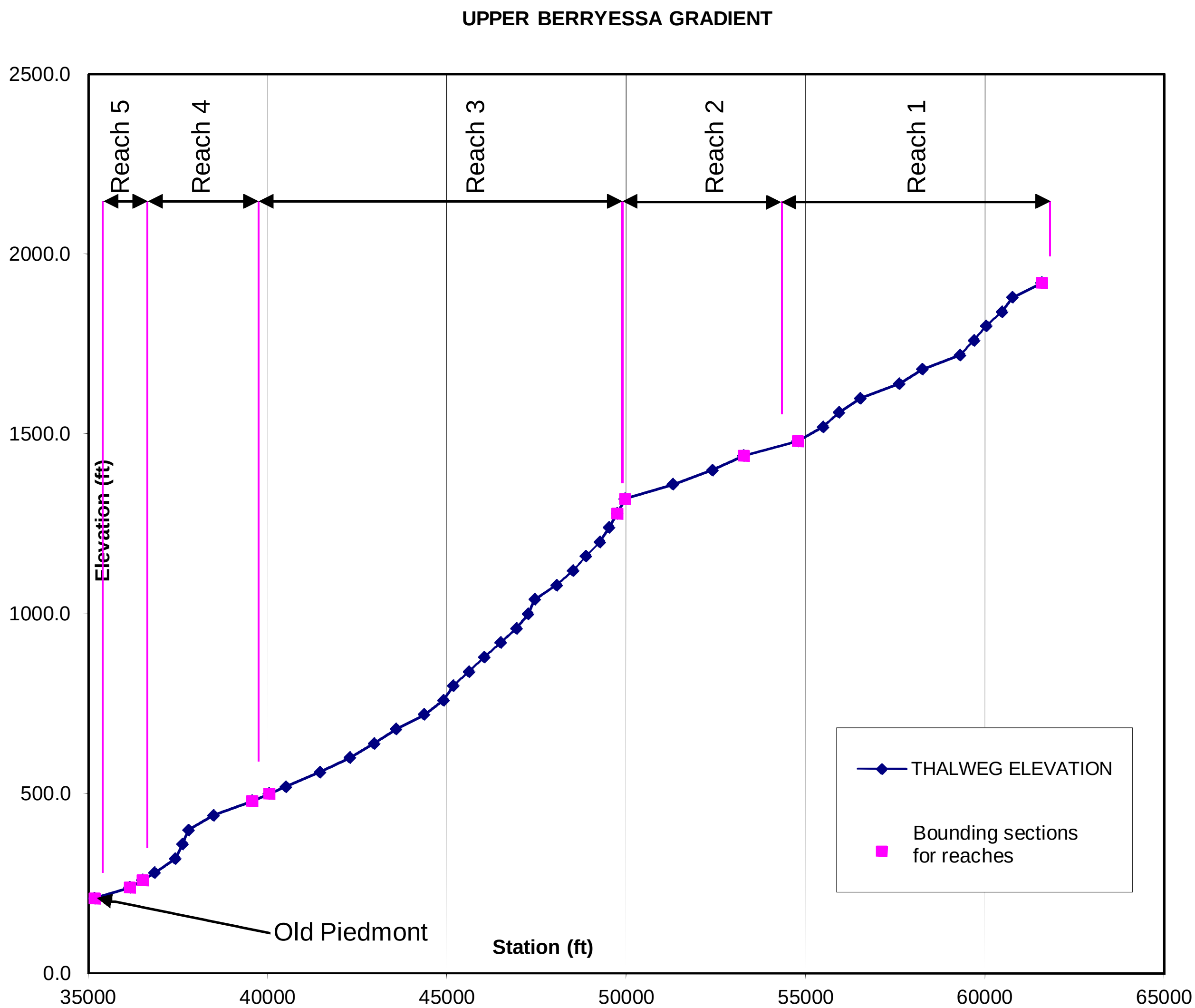

Figure 2-5 Berryessa Creek Profile from Old Piedmont Road to Headwaters

2.1.3 Channel Geometry

Within the project area, Berryessa Creek occupies a constructed channel that is heavily constrained by bridges, bank protection, channel lining and other constructed features. Thus channel dimensions are more a result of these influences as opposed to natural geomorphic processes. For description of the channel geometry, the project area was divided into eight reaches. During the analysis of the preliminary array of alternatives it was found that the portion of the project between Old Piedmont Road and I-680 was not justified and those portions of the project were removed from the final alternatives. Nevertheless, the six reaches between Old Piedmont Road and I-680 are described here to ensure continuity with the preliminary analysis completed prior to 2009. Descriptions of each reach are provided below. Additional details on channel cross sections can be found in the *Part I: Hydraulic Analysis of Alternatives* and *Part IV: Design and Cost of Alternatives* in this engineering appendix.

Calaveras Boulevard to Montague Expressway (Sta 138+03 to 217+38) – This reach is a straight, excavated earthen channel. It appears to have originally been excavated as a trapezoidal channel, but in some areas erosion and incision have resulted in the formation of steep, near vertical banks. The channel averages on the order of 10 to 12 feet in depth. The top width varies from a narrow 35 feet near the railroad trestle to on the order of 50 feet in other locations. The channel conveyance capacity ranges from 1,300 to 2,500 cfs.

Montague Expressway to I-680 (Sta 217+38 to 255+75) – This is another section of constructed trapezoidal earthen channel; with the exception that the channel bed and banks have been lined with concrete through the three 90 degree bends in this reach. The channel is approximately 40 feet wide with a depth of 7 to 8 feet. The conveyance capacity ranges from 800 to 1,500 cfs.

Upstream of the project area, the channel configuration and constraints vary significantly:

I-680 to Cropley Avenue (Sta 255+75 to 275+69) – This reach of Berryessa Creek is contained in a trapezoidal concrete channel with a top width on the order of 40 feet and a depth of 10 feet. These dimensions include the upper one to two feet of earthen material that continues to form channel sideslopes above the concrete. This segment of Berryessa Creek can contains approximately 2,800 cfs.

Cropley Avenue to Morrill Avenue (Sta 275+69 to 285+93) – This reach is a constructed trapezoidal, earthen channel with 2:1 sideslopes. The beds have been protected with concrete. The top width is on the order of 45 to 50 feet and the depth is typically 8 feet. The channel can contain flows up to approximately 1,500 cfs. The Cropley Avenue Bridge is a major constriction that creates a backwater upstream through much of the reach.

Morrill Avenue to Sierra Creek (Sta 285+93 to 292+00) – This reach is a combination of constructed channels. The downstream portion is a rectangular concrete channel with a 20 foot top width. The middle section is a trapezoidal channel with a gravel bed and banks protected by sacks filled with concrete. The top width is approximately 40 feet. The most upstream section is a drop structure that continues with banks protected by sacks filled with concrete, but has a concrete channel bottom. The top width of this segment is also approximately 40 feet. All three sections have depths on the order of 8 to 10 feet and contain flows up to approximately 1,500 cfs.

Sierra Creek to Piedmont Sediment Retention Basin (Sta 292+00 to 338+04) – This reach is referred to as the Greenbelt Reach. It contains the only section of channel that is not an excavated section constructed on an engineered alignment. The reach has only minor influences from bridges within its boundaries, with one pedestrian bridge crossing the channel without restricting it. The 20 to 30 foot wide channel varies from about 3 to 6 feet in depth. Portions of the channel have incised some, but banks remain stable due to vegetation and the silt and clay content which was reported to be roughly 50 percent (NHC 1990). Though the channel is free to meander within the 100 to 150 foot wide floodplain, the channel is fairly straight at a sinuosity of 1.06. The channel capacity is more representative of a natural stream section in this reach than in other reaches with a bankfull capacity of approximately 500 cfs. The treed floodplain, which in some areas has berms and fill to help contain floods, can convey on the order of 1,300 cfs before flows breakout. Two tributaries, Crosley Creek and Sweigert Creek, enter in this portion of Berryessa Creek.

Piedmont Road Sediment Basin to Cropley Avenue (Sta 338+04 to 344+67) – This reach is comprised of two features. The downstream 250 feet is a sediment basin and the upstream 410 is a 12-ft by 7-ft concrete box culvert. To form the sediment retention basin, the channel has been widened and the banks protected to create an area to slow velocities and reduce shear stresses in order to collect upstream sediments. The sediment is then removed with construction equipment from the basin. The channel widens to 80 feet in the basin and has a depth that varies from 9 feet at the upstream end to about 6 feet as the basin transitions to the Greenbelt Reach. Santa Clara Valley Water District (SCVWD) records indicate that on the average nearly 527 cubic yards of sediment (see Table 2-1) are removed from the Piedmont Sediment Basin per year. The 410 foot long culvert that passes beneath the intersection of Piedmont Road and Cropley Avenue experiences deposition of coarse bed load from the build-up of material in the sediment retention basin. The basin will convey flows on the order of 1,500 cfs, but the culvert capacity is limited to passing approximately 900 cfs. The culvert capacity is often further restricted by sediment deposition within the culvert that can reduce the capacity to approximately 600 cfs or less.

Cropley Avenue to Old Piedmont Road (Sta 344+67 to 351+70) – This is an incised channel section with a width of approximately 40 feet and a depth of 10 feet. The channel banks in this reach have considerable gravel and small cobbles, though there is sufficient finer material for cementation to hold the banks near vertical. The channel capacity is approximately 1,500 cfs.

2.1.4 Current and Historical Channel Planform

The channel planform in the project area has undergone large changes since the middle of the 19th century. These are discussed in detail by NHC (2001) and summarized in this section. Of importance to understanding of the current conditions and the influences on the development of the flood control project is a comparison of the historic and current conditions. Before development, Berryessa Creek and its major tributaries flowed onto the alluvial fan for several thousand feet before spreading into distributary channels or infiltrating to the point that they were no longer shown on maps. As development increased, the streams were channelized to provide flood control and to supply irrigation water. It is also indicated that subsidence in the Santa Clara Valley may have contributed to the down fan progression of the defined stream channels.

By 1943, maps indicate that Berryessa Creek joined Penitencia Creek about 2 miles upstream of their current confluence. Significant realignment occurred between 1953 and 1961 when the creek was realigned to flow northward. This realignment placed the channel within its general flow path from the current I-680 crossing to Penitencia Creek. As a result of this realignment, the channel gradient was reduced from close to1 percent to less than 0.5 percent. The prior west flowing alignment was directly down the fan gradient whereas the realignment flows across the fan. This is the reason for the abrupt reduction in gradient previously discussed for the reach mentioned from I-680 to the Montague Expressway. In 1976 the downstream-most portions of Berryessa Creek was realigned by the SCVWD as part of a flood control program. The current alignment from the fan apex to I-680 is close to that identified for 1943. The uppermost section of Berryessa Creek, from the apex to the middle of the Greenbelt Reach, is currently in the same general location as identified in 1899 maps.

2.1.5 Upper Watershed Site Inspection

An inspection of the Berryessa Creek watershed upstream of Old Piedmont Road was performed in August 2004. Participants in the field trip included representatives of the Sacramento District and Tetra Tech. The purpose of the field trip was to observe watershed and stream conditions that influenced sediment production and yield in order to develop potential strategies to reduce downstream sediment loading. More specifically the inspection was conducted to identify sediment sources, watershed processes controlling erosion and sedimentation, potential locations for sediment control facilities and the potential for land management activities to control sediment supply.

There were five distinct areas or zones observed in the stream and adjacent watershed. In the upper most 1.3 miles (Reach 1, upstream of the 1,480 foot contour), the creek is of moderately steep gradient averaging 6.5 percent and has a bed comprised of a wide range of material from gravels and cobbles to fines. The channel may be incised in some areas by several feet. There did not appear to be a high transport rate of the larger bed material (gravel and cobble) as there were few depositional bed features and there was a significant amount of finer material in the bed and heavy vegetation on the banks (Photo 2.1). On the hillsides, some minor gullying was observed where flow had been concentrated by roads or trails, but in the small gullies there were only a scattering of coarser materials so that it does not appear that this process is a significant source for coarser sediments in the upper portion of the watershed.

Photo 2.1 Typical Channel in Reach 1, Heavy Vegetation on Banks

The second segment of the channel (Reach 2) is relatively low gradient, particularly considering its location high in the watershed. This flatter section extends for approximately one mile at an average gradient of 3 percent, from the 1,480 foot contour on downstream to the 1,320 foot contour. Though the gradient flattens, the channel still has an incised appearance in areas. A significant depositional area of coarse material was not observed in this reach. This implies that the sediment production, of coarser materials is not high in the upper reach, otherwise the material would deposit in the area of reduced slope. The bed was comprised of sands and silts in portions of this reach, with only a scattering of angular gravels and cobbles (Photo 2.2). These larger materials may have fallen into the channel from the adjacent banks. In some areas where the bank material was exposed, there was a fairly heterogeneous matrix of material ranging from fines to small cobbles.

Photo 2.2 Typical Channel in Reach 2, Low Gradient

The third segment (Reach 3) of the upper channel starts as the stream gradient steepens and the channel becomes confined by steep hillsides. The bed material becomes dominated by gravels, cobbles and boulders with some bed rock outcroppings (Photo 2.3). The gradient was estimated at 8 percent for this reach which extends for approximately 2 miles to the 500 foot contour. Passage down the creek became difficult, so the inspection team walked along the hillside on the north side of the channel. At the several locations where the team returned to the creek bed, it was evident that the channel was capable of transporting materials up to boulders of over a foot in diameter. At several locations, bedrock was exposed in the channel and small falls were created. Though the watershed is very steep in this reach, the only landslides were observed near the downstream boundary of this reach. The south side of the valley wall is heavily forested while the north side is dominated by shrubs and grasses, except for a strip along the very bottom of the valley near the channel.

Photo 2.3 Typical Channel Section in Reach 3, Gradient of 8 Percent

Reach 4 begins where the stream enters what was referred to in previous reports (USACE 1993 and NHC 2001) as the canyon reach. The reach extends for approximately 0.6 miles at an average gradient of 8 percent with a short steep section of over 15 percent in the center of the reach. The most striking feature in this reach are a number of larger landslides that start hundreds of feet up on the hillside and continue down to the creek (Photo 2.4). These features are the largest concentrated sediment sources observed. The creek bed in this area is dominated by coarse material ranging from gravels and cobbles up to boulders on the order of 4 feet in diameter and greater. There is evidence that at times, the channel has transported debris torrents or flows. The formation comprising the surficial geology in this portion of the watershed is more susceptible to erosion and mass wasting than further upstream (Photo 2.5). This condition is further influenced by the Hayward Fault zone. The reduction in vegetative cover as elevation and rainfall decreases may also be a factor.

Photo 2.4 Mass Wasting Directly into Creek near Upstream Limits of Reach 4

Photo 2.5 Landslide Scarp on North Valley Wall in Reach 4 (Canyon Reach)

Reach 5 is a transition zone from the steeper upper watershed to the much flatter alluvial fan. The average gradient through this 0.3 mile reach is 4 percent. The channel bed in this reach is still comprised of material ranging from gravels to large boulders (Photo 2.6). Most or all of the larger boulders generated upstream appear to be deposited in this reach and do not cross Old Piedmont Road.

Photo 2.6 Typical Reach 5 Channel in Transition from Uplands to the Alluvial Fan

2.1.5.1 Implications of Watershed Inspection

Based on the observations during the site visit, control of sediments from the upper two segments (Reaches 1 and 2) of the watershed would have minor influence on delivery of coarse sediments (gravel and cobbles) to the reaches below Old Piedmont Road since it appears very little of this size material would make it through the flatter gradient of Reach 2. Sands and finer sediments may be produced in these areas, but their relative contribution would appear to be smaller than the portions of the watershed further downstream.

Based on the coarse bed material and steep gradient in Reach 3, a significant amount of gravel and cobble can be transported through this reach. However, no large point sources were identified. The team did not walk this portion of the creek bed so it could not be observed if there were large areas of bank erosion or contributions of sediments from point sources along the creek. This statement is based mainly on the lack of gullies crossed in walking along the north side of the valley wall and no visual identification of larger landslides on either the north or south valley wall. Construction of a sediment retention facility in this reach would be difficult due to the limited access and the small amount of storage volume per foot of structure height because of the steep channel gradient and steep confining valley walls.

Reach 4, the 0.6 mile length of the creek and associated watershed above Old Piedmont Road, appears to be the most significant area of sediment production. This is the area that several large point sources of sediment were identified, in the form of landslides in which feed directly into the creek. If a sediment retention or trap facility were to be constructed, it would appear that the best location would be in Reach 5 as the gradient decreases and the area adjacent to the channel increases. This area would control the large contribution of sediment from Reach 4. Lastly, this area has the best access for construction and maintenance.

In terms of land management, much of the upper watershed is grazed. There are a few residences, mainly along the watershed divide. The primary road serving the watershed travels near the watershed divide and in the majority of locations is in the adjacent watershed. There did not appear to be significant erosion problems created by any of these watershed disturbances. For example, there were no gullies observed as the result of concentration of flows from roadside drainage or from residential development. Likewise, there was no evidence of significant rilling or gullying occurring on the grazing lands or of trampling of streambanks by livestock. However, the influence of grazing was quite apparent with numerous trails contouring the hillsides and some locations with hillsides covered with hoof imprints left from the rainy season. Any control measures adopted to limit grazing activities along the channel banks would primarily reduce the fine sediment yield.

2.2 Summary of Sediment Transport Conditions

This section presents information on the current sediment transport conditions for the project area and upstream reaches that were presented in previous studies. The sediment removal history is also reviewed. The results of the hydraulic analysis for the with-project alternatives are utilized to qualitatively determine changes in sediment transport and removal requirements that would be induced by the project.

2.2.1 Previous Studies - Sediment Budget and Modeling

Previous analyses of the sediment budget (HMC 1990), geomorphology (NHC 2001) and sediment transport (NHC 2003) for the without-project condition of Berryessa Creek indicated two potential problems. The first was potential areas of deposition and the second was potential areas of degradation.

2.2.1.1 1990 Sediment Budget Analysis

An overall estimate of the sediment yield for Berryessa Creek was developed by NHC (1990). The results of this analysis indicated the following sediment yields:

Berryessa Creek at Old Piedmont Road	=	9,900 tons/year
Sweigert, Crosley, and Sierra Creeks	=	1,900 tons/year
Piedmont Creek	=	700 tons/year
Arroyo de los Coches	=	3,200 tons/year

The values provided for the tributaries are at their confluence with Berryessa Creek. The total yield is 15,700 tons/year. If a dry unit weight of 100 lbs/ft^3 is assumed for sediments, this represents 11,600 cubic yards per year.
The sediment budget performed by NHC (1990) estimated the mean annual inflowing sediment load at Calaveras Boulevard to be 9,200 tons/year or 6,800 cubic yards per year. This budget was based on deposition of 6,700 tons/year of sediment between Piedmont Road and Calaveras Boulevard. The study utilized a value of 5,000 cubic yards per year of sediment removal upstream of Calaveras Boulevard.

It should be noted that the 1990 study used a value of 23,800 cubic yards of sediment removed in 1983 between Sierra Creek and Calaveras Boulevard.

2.2.1.2 2001 Geomorphology Study

In 2001 NHC updated the 1990 sediment budget analysis (NHC 2001). One major change aside from the additional sediment removal data available was that the large value of 23,800 cubic yards of sediment removed in 1983 between Sierra Creek and Calaveras Boulevard was not included. If this large volume of removal is not included, the average annual rate for the 10-year period referenced in the 1990 Sediment Budget Analysis (NHC 1990) would be 2,620 cubic yards per year or 3,200 tons/year (NHC assumed 90 lbs/ft^3 for deposited sediments). This change in assumptions and additional sediment removal data resulted in the sediment budget resulting in 12,400 tons/year of sediment passing Calaveras Boulevard as opposed to the 9,200 tons/year as indicated in the 1990 study.

2.2.1.3 2003 Sediment Transport Modeling

In 2003 estimates of sediment yield and budget were developed by NHC based on an HEC-6T sediment transport analysis (NHC 2003). The sediment yield was computed by integrating the HEC-6T simulated bed material load yields for the single storm events to determine average annual yields utilizing the method described by Mussetter *et. al.* (1994). This resulted in an average annual bed material yield at Old Piedmont Road of 2,500 to 3,000 tons per year. The overall budget identified a total of 170 tons per year of net erosion from the reach, indicating this reach is currently slightly degradational. This minimal amount of degradation translates into an average of 0.05 inches per year if the total volume were to be spread out over the entire reach. The sediment budget presented in the 2003 report did not indicate it accounted for sediment removal that takes place at several locations throughout the reach. The budget also did not provide an indication of the simulated tributary inflows and how or if they were accounted for in the budget.

2.2.1.4 Analysis of Previous Studies

If the 9,900 tons per year average annual sediment yield at Old Piedmont Road computed in the 1990 Sediment Budget Analysis is assumed to be 35 percent bed material load (sand, gravel and cobble) and 65% wash load (silts and clays), the resulting average annual bed material supply at Old Piedmont Road is 3,500 tons. This is in fairly close agreement with the 2003 HEC-6T Sediment Transport Study which indicated an average annual upstream loading on the order of 2,500 tons per year. In terms of the sediment balance in the reach, the HEC-6T modeling by NHC indicated a slight degradational trend. However, the modeling did not appear to include the sediment removal in the analysis. Accounting for sediment removal increases the degradational trend by several thousand tons per year. An overall degradational trend is supported by comparisons of the 1968 and 1998 channel thalweg profiles in the 2001 Geomorphic Study (NHC 2001). Comparison of these profiles indicates that the 1998 profile is at or below the 1967 profile throughout the project area. Continued sediment removal prevents the areas of deposition from being revealed on the profile comparison.

Because of the highly manipulated nature of the Berryessa Creek channel within the project area, its ability to transport sediment varies widely. Though there are segments of considerable deposition that require sediment removal to maintain flood conveyance capacity, there are areas with higher sediment transport capacity that result in channel degradation. This is supported by the comparison of the 1967 and 1998 thalweg profiles presented by NHC in the 2001 Geomorphic Study. The 2003 HEC-6T sediment modeling results show similar behavior with a slight overall trend for degradation, but a mixture of aggradation and degradation scattered throughout the project area.

The 2003 HEC-6T model results indicated that the bed material load from a single 1% chance exceedance event would be on the order of 13,000 tons at Old Piedmont Road, which is on the order of four to five times the estimated average annual bed material loading. During a 1% chance exceedance event, the maximum predicted aggradation is over 4 feet at the Piedmont/Cropley culvert and over 2 feet just upstream of the Ames Avenue Railroad trestle. At all other locations the aggradation is on the order of one foot or less. The maximum predicted degradation is 2 feet in the Greenbelt Reach just downstream of the sediment basin and just over one foot about 500 to 1,000 feet upstream of Los Coches Street. Based on these results the modeling indicates a mixture of aggradation and degradational areas. Though the actual historic profiles indicate primarily equilibrium or degradational reaches, the model did not appear to account for the sediment removal in the aggradation areas. If all sediment deposits indicated by the model results are removed, the required sediment removal predicted by the HEC-6T model would be on the order of 3,700 cubic yards per year. A further discussion of actual sediment removal history is presented in the next section.

2.2.2 Sediment Removal History

The SCVWD performs removal of sediment on an as needed basis to maintain the conveyance capacity of Berryessa Creek throughout the project area and upstream reaches. The two concentrated areas of removal upstream of the project area are the sediment retention basin below Piedmont Road and the reach between the Sierra Creek confluence downstream to Cropley Avenue. Additionally, sediment is removed at various locations throughout the project area. Table 2-1 presents the reported maintenance records of sediment removal from five reaches within the Berryessa Creek channel. The sediment removal for the study area between Old Piedmont Road and I-680 is divided into two reaches, the sediment retention basin below Piedmont Road and the area from Sierra Creek to Cropley Avenue. The sediment removal for the study area downstream of I-680 is also subdivided into two areas; I-680 to Montague Expressway and Montague Expressway to Calaveras Boulevard. The final reporting reach downstream of Calaveras Blvd and is outside of the project area.

Based on 33-years of maintenance records from 1977 to 2011 the most concentrated area of sediment deposition in the study area is at the sediment retention basin below Piedmont Road. In this several hundred foot long reach, an estimated average annual removal of 527 cubic yards occurs. This is the highest removal at any location in the study area and also represents the shortest stream reach of all the removal areas. The next highest sediment removal area is Sierra Creek to Cropley Avenue. In this 1,600 foot long reach, the estimated average annual removal is 525 cubic yards. In the 3,600 foot long reach from I-680 to Montague Expressway, the level of sediment removal is slightly less than the two upstream sites at 430 cubic yards per year. The lowest annual sediment removal is found in the downstream-most reach in the study area, from Montague Expressway to Calaveras Boulevard, an annual average of 205 cubic yards is removed in its 7,700 foot length.

Table 2-1 Summary of SCVWD Sediment Removal Maintenance Records on Berryessa Creek (NHC 2001 and SCVWD)

Year	Removal in Deposition Areas (cu. yd.)					Total (cu. yd.)
	DS of Calaveras	Montague to Calaveras	I-680 to Montague	Cropley to Sierra Creek	Piedmont Sed. Basin	
1977	0	0	0	0	0	0
1978	0	0	0	0	0	0
1979	0	0	0	0	0	0
1980	0	0	0	0	0	0
1981	4,210	4,100	0	0	0	8,310
1982	23,510	0	2,890	0	0	26,400
1983	0	0	0	0	0	0
1984	19,500	0	0	0	0	19,500
1985	14,352	0	1,136	1,137	1,137	17,762
1986	460	1,320	0	3,260	900	5,940
1987	9,820	800	250	0	0	10,870
1988	0		0	10	2,724	2,734
1989	13,330	400	0	432	0	14,162
1990	10,520	0	0	0	0	10,520
1991	4,066	0	0	0	300	4,366
1992	0	0	0	0	0	0
1993	2,800	0	0	2,500	1,250	6,550
1994	0	0	0	0	0	0
1995	0	0	0	0	0	0
1996	0	0	0	0	5,600	5,600
1997	30,000	0	0	700	810	31,510
1998	0	0	0	3,850	1,000	4,850
1999	1,250	0	8,850	0	0	10,100
2000	0	0	0	0	1,300	1,300
2001	7,189	0	0	3,165	1,525	11,879
2002	0	0	0	0	0	0
2003	4,640	0	0	0	0	4,640
2004	7260	0	20	0	450	7,730
2005	0	0	0	0	0	0
2006	0	90	0	1,744	930	2,764
2007	6,320	67	500	0	0	6,887
2008	0	0	964	0	0	964
2009	0	0	0	0	0	0
2010	0	0	0	1,040	0	30,040
2011	34,000[1]	0	0	0	890	34890
Average Annual	5,521	199	417	509	537	7,179
Totals	193,227	6,777	14,610	17,838	18,816	251,268

Note: 1. Maintenance has been deferred for the reach downstream of Calaveras from 2008 to present pending reconstruction of the reach by SCVWD. The current estimate by the SCVWD Water Operation Staff of 34,000 cubic yards of sediment in this reach is used to account for this deferred maintenance. (SCVWD 2011a)

The sediment deposition basin below Piedmont Road was developed to collect sediment as the channel leaves the upstream watershed and flows onto the alluvial fan. At the Piedmont Road sedimentation basin, the channel gradient has been reduced and the width increased to form the basin. In the Sierra Creek to Cropley Avenue reach, a combination of drop structures, energy dissipaters and restrictive bridges, as well as the possibility of supply of additional sediments from the Greenbelt Reach and Sierra Creek, result in an area of concentrated deposition. Below I-680, the overall gradient dramatically decreases by a factor of 2 to 3 compared with the reach from Cropley Avenue to I-680. As a result of this gradient reduction, the reach is subject to aggradation in areas where the channel widens or flows are backwatered upstream of restrictive bridges.

The results of the 2003 Sediment Transport Modeling were compared to the maintenance records sediment removal results presented in Table 2-1. In order to compare the two analyses, the results for the SCVWD sediment removal reaches reported in Table 2-1 were developed from the 2003 HEC-6T modeling. Note that the reported HEC-6T model estimated volumes do not include some areas of lesser deposition not included in Table 2-1, resulting in the total estimated average annual deposition for the sediment removal reaches not equaling the 3,700 cubic yards per year reported for the study area in the previous section. The resulting average annual sediment removal volumes for the SCVWD sediment removal reaches predicted in the HEC-6T model are listed in Table 2-2.

Table 2-2 Comparison of SCVWD Sediment Removal Records and NHC 2003 HEC-6T Sediment Transport Modeling

Sediment Removal Reach	Average Annual Sediment Removal Estimates (Cubic Yards per Year)		
	SCVWD Maintenance Records	2003 NHC HEC-6T Modeling	Percent Difference from SCVWD Records
Piedmont Sediment Basin	527	890	69%
Sierra Cr. to Cropley Avenue	525	390	-26%
I-680 to Montague Expressway	430	720	67%
Montague Expressway to Calaveras Boulevard	205	860	319%
TOTAL	**1,687**	**2,860**	**69%**

The 2003 Sediment Transport Modeling results reported in Table 2-2 are approximately 70 percent higher than those reported by SCVWD maintenance records for the total study area and of the two removal reaches. The only reach underestimated by the 2003 HEC-6T modeling in comparison to maintenance records is from Sierra Creek to Cropley Avenue where the HEC-6T results indicate 390 cubic yards and the maintenance records identify 525 cubic yards per year. In contrast, the HEC-6T model overestimates the required sediment removal in the Montague Expressway to Calaveras Boulevard reach by over 319%.

It should be noted that significant sediment deposition requiring removal occurs in the 8,500 foot reach from Calaveras Boulevard downstream to the Penitencia Creek confluence. This reach is tidally influenced and therefore sediment deposition is expected. In the GDM (USACE 1993), based on removal records from 1981 to 1990, the removal in this reach was equal to the total removal for all upstream reaches averaging 5,000 cubic yards per year.

Correspondence from the SCVWD indicated sediment removal operations has been performed downstream of Calaveras Boulevard eight times since 1990 with removal volumes ranging from 1,250 cubic yards in 1999 to 30,000 yards in 1997. In addition, recently sediment maintenance activity has been deferred for this reach because of pending reconstruction activity by SCVWD. To account for the sediment deposition in the reach from 2008 to present, the SCVWD Water Operation Staff has estimated that the volume of sediment that would have been removed for routine sediment operations in the reach is 29,000 cubic yards (SCVWD 2011a). The addition of the sediment removal activity since 1990 results in an average annual sediment removal of 4,683 cubic yards per year for Berryessa Creek from the confluence of Penitencia Creek to Calaveras Boulevard.

In evaluating the influence of with-project alternatives, consideration must be given to the portion of Berryessa Creek downstream of the project limits. Two important aspects of the sediment balance need to be incorporated into the overall project evaluation. First, if additional sediment is generated from bank erosion or bed degradation in the project area, if it is not deposited in the project area, most of the sediment would be deposited in the reach below Calaveras Boulevard. Second, any reduction in maintenance requirements that results from increasing sediment transport capacity within the project area will pass sediment through the project area, but will result in increased deposition in the reach below Calaveras Boulevard.

CHAPTER 3: WITH-PROJECT CONDITIONS

This chapter applies the information from the existing conditions assessment of geomorphology and sediment transport investigations to identify design considerations and issues to be addressed in the with-project alternatives. Results of the hydraulic analysis of the without and with-project alternatives are compared to qualitatively identify potential channel responses. The information is applied to identify recommendations as to potential modifications or refinements of the with-project alternatives. Sediment management features between Old Piedmont Road and I-680 are not part of the current project but are under consideration by others. These features are included herein for discussion purposes as the sediment supply through the upstream reaches affects the configuration of sediment management features in Alternatives 2A/d, 2B/d and 4B/d downstream of I-680.

3.1 Design Issues and Considerations

The following section identifies the issues or considerations, and then provides recommendations as to how they may be addressed in the alternatives. The general categories of issues to address are:

- Management of coarse sediment
- Minimize aggradation and degradation
- Provide opportunities for environmental enhancement

3.1.1 Management of Coarse Sediment

The Berryessa Creek Project Area extends from I-680 to Calaveras Boulevard and lies within an alluvial fan. Alluvial fans are created by sediment deposition as streams carrying large sediment loads exit the steep confined channel of the uplands and meet the lower gradient unconfined valley. As a result, sediment deposition is an inevitable process on an alluvial fan and any channel improvements must recognize this behavior. On the Berryessa Creek fan, at some point, between the apex of the fan and the Bay, all but the finest sediments will be deposited. Since the gradient decreases in the downstream direction along the fan, and the ability to transport sediment decreases along with it, the larger sediments are deposited furthest upstream.

Deposition in the project area currently requires on the order of 1,046 cubic yards per year of sediment between Old Piedmont Road and I-680 and 616 cubic yards per year of sediment downstream of I-680 be removed. Additional sediment deposits are also removed downstream of the project area. Even if a concrete channel that confined all the flow and maximized velocities and shear stresses were installed, though the coarse sediments would be conveyed further, they would either deposit in the lower gradient project area downstream of I-680 or in the tidally influence reach further downstream. Therefore at some point along Berryessa or Penitencia Creek, the sediments become a maintenance issue because removal is required to maintain flood conveyance capacity and prevent the eventual plugging of the channel. Coarse sediment management approaches to be considered include reducing the supply of sediment and promoting sediment deposition in areas that will not induce flood problems and are readily accessible to perform periodic sediment removal.

3.1.2 Reduction of Coarse Sediment Supply

Coarse sediment supply is generated primarily upstream of the project on the mainstem of Berryessa Creek and passes through the bridge at Old Piedmont Road. Additional quantities of sand and gravel are supplied by the larger tributaries and some sediment may be generated from channel degradation and bank erosion within the project area. Inspection of the upland watershed and information contained in past studies indicate that the majority of coarse sediment is generated in the lower steep canyon reaches (Reach 4) of Berryessa Creek as a result of mass wasting and erosion of the steep hillsides immediately adjacent to the creek. Because of the scale of these sources and the fact that they are a result of natural process and conditions, including the presence of active fault zones and unstable geologic formation, controlling the coarse sediment supply at its source is not practical.

Another option would be to create a sediment retention basin upstream of Old Piedmont Road in the transition zone from the steep canyon to the alluvial fan. This is the zone that the large boulders that may be transported in debris torrents and flows are deposited in. Additionally, smaller boulders and cobble are also deposited in this area. The 1989 Authorized Plan and 1993 GDM (USACE 1993) included a sediment basin at this location with a capacity of 17,000 cubic yards which exceeds the volume of sediments deposited in a 1% chance exceedance event (12,000 cubic yards) plus the average annual sediment deposition (3,000 cubic yards).

The difficulty with such a large basin is that it would trap nearly all of the sediments from sand size and larger. This would result in the “hungry water” released from the sediment basin picking up sediments further downstream which would result in bed and bank erosion. This would likely cause the channel through the Greenbelt Reach to become incised and less connected to its floodplain. In the case of the channel design presented in the 1993 GDM, a concrete channel would be installed downstream of Old Piedmont Road. The concrete channel would have prevented bed degradation and bank erosion. However, with the “natural” channel bottom being proposed in the current with-project alternatives, the bed would be subject to degradation. Thus installation of a large sediment basin above Old Piedmont Road does not appear to be compatible with the implementation of a project with an alluvial bed. Given the limitations of a sediment basin at this location, a debris trap is considered as a possible future refinement of the GDM design. For the purposes of this study, the sediment basin upstream of Old Piedmont Road was analyzed as designed in the 1993 GDM since this was a component of the Authorized Project which needs to be analyzed as designed.

3.1.3 Debris Torrents and Flows

Based on site observations and past reports (USACE 1993 and NHC 2001), the potential for transport of large boulders in the form of debris torrents and flows exists. It appears that this material is transported almost as far as the Old Piedmont Road crossing and could cause problems with the culvert. To reduce the possibility of plugging the culvert, which could result in the flows breaking out of the channel, an installation of a debris fence or other permeable structure designed to strain debris flows will be investigated upstream of Old Piedmont Road during the next phase (design of the selected plan) of the GRR. Such a structure would catch the larger material but allow passage of the majority of cobble and finer material. The structure would have little influence on normal flows. By only catching the larger material and debris, the volume of storage behind the structure is much smaller than for a sediment basin. Additionally, since it passes the majority of the sediment load, it does not have the potential to induce channel degradation downstream. The structure will need access for removal of trapped material; however, removal will only need to be performed after large events that mobilize boulders. The inclusion of the debris fence would not affect plan selection.

3.1.4 Coarse Sediment Management within the Project

Currently, coarse sediment is managed in the project by periodic removal of deposits. In most cases, sediment is removed from locations within the project area on an as-needed basis. The sediment retention basin upstream of the project area at Piedmont Road has been designed to facilitate sediment removal. This basin collects bed material load by providing a wide area with reduced flow velocity and shear stress. The capacity of the basin is on the order of 1,000 to 1,500 cubic yards. A significant problem with the basin is that once sediments start depositing in the basin, they quickly create a backwater that causes sediment to deposit in the 410 foot long culvert immediately upstream. This reduces the flood conveyance capacity of the culvert, which can result in flows breaking out upstream of the culvert at much lower return periods and increasing the frequency of flooding. In addition, it is extremely difficult to remove deposits from the culvert due to the limited workspace and clearance.

Several modifications should be considered for the basin to improve its performance. Potential modifications include regrading the basin to have a steep slope immediately downstream of the culvert outlet. This would provide sediment storage below the culvert invert and reduce the tendency for deposits to build up in the culvert. Additionally, the culvert invert could be altered to have a V-bottom. This would help concentrate flows and increase the transport capacity during low flows. Another potential option is to move the basin a short distance downstream so that there is some distance between the basin and the culvert outlet. The area between the two features should have a steep slope to prevent backup of deposits into the culvert. It is noted that increasing the storage volume of the basin may not be a good option. A significant increase in the volume would increase the trap efficiency which could induce channel degradation and incision in the Greenbelt Reach.

Accommodating the steep chute below the culvert or the shifting of the basin further downstream would require lowering the basin and possibly alteration of some of the channel in the Greenbelt Reach. Changes to the channel in the Greenbelt Reach should be analyzed carefully and kept to a level that does not create problems with the stability of this reach. Potential problems that would have to be mitigated would be reduced stability after disturbing the vegetation on the banks and increased flow confinement if the channel was lowered.

In addition to improvements to the Piedmont sediment retention basin, additional coarse sediment management might be provided by creation of locations that were designed to conduct sediment removal operations. This would involve providing access to the channel bottom and possibly altering channel hydraulics to encourage sediment deposition. Based on historical sediment removal, likely locations would be between the Sierra Creek confluence and Cropley Avenue crossing and between I-680 and Montague Expressway. Sediment transport modeling of these facilities would be necessary to ensure that they function properly and do not trap so much sediment that downstream degradation problems are created. Additionally, locations for the facilities should be determined after sediment transport modeling of the with-project condition since the channel alterations under the with-project condition may alter the locations most prone to sediment deposition.

A high-flow bypass culvert running beneath Cropley Avenue is being considered by the SCVWD to reduce flooding in the Greenbelt reach. Detail planning for the SCVWD bypass plan has not been completed at the time of this study. Approximate sediment management implications are presented in this report and will be added to future design reports. The bypass alternative was only considered for the design of Alternatives 2B/d and 4/d.

3.1.5 Minimize Channel Bed Aggradation and Degradation

Berryessa Creek has areas that experience aggradation and others that have experienced degradation. If not properly accounted for, alteration of the system for flood control has the potential to increase either or both of these processes at various locations within the project area.

3.1.5.1 Flow Confinement

Confinement of higher flows to a limited area by excavation of a larger channel or construction of levees increases shear stresses which can mobilize larger sediments and increase transport rates. As a result, the flows erode sediments from the bed to satisfy the increase in sediment transport capacity. These sediments may be deposited downstream when the flows reach a portion of the channel where the hydraulic conditions become less severe. Evaluation of the Berryessa Project alternatives needs to account for this potential since much of the project involves measures that increase the flow confined to a main channel.

Sediment transport analysis and modeling should be conducted to refine the design of the selected alternative to assess areas where this may be a problem. If such locations are identified, then the channel dimensions need to be modified to reduce the potential for degradation. If this cannot be done, while maintaining flood control objectives, then the inclusion of grade controls to limit future degradation should be considered.

3.1.5.2 Channel Widening

In some cases excavation of a wide channel to create sufficient cross-sectional area to pass the design flows can actually result in reducing sediment transport capacity for smaller events. Though very large floods pass a greater amount of sediment on a single event basis, smaller flows, owing to their greater frequency of occurrence, are typically responsible for the greatest portion of sediment transport over the long term. The flood responsible for the greatest portion of sediment transport is referred to as the dominant or formative discharge and often ranges between the 20- to 75% chance exceedance events. Therefore, a reduction in sediment transport capacity at the lower return period floods, by spreading across the wider channel bed, may off-set the increase in sediment transport capacity created by confining the larger floods to the enlarged channel. Depending on the magnitude of the changes, the two factors may offset creating a condition of dynamic equilibrium or the change may be so large as to shift the channel into an aggrading mode. In some widened channels, alternate bars may form during low flows that become vegetated and cannot be removed at higher flows in some reaches. Though the channel might have the capacity to transport the sediment stored in the bars, the vegetation in some reaches prevents them from becoming scoured and they may need to be removed as part of a maintenance program. Since portions of the Berryessa Creek channel are widened, this behavior is also a possibility.

Sediment transport analysis and modeling for the selected alternative should identify any areas where channel widening is causing excessive degradation. If such locations are identified, the design should determine whether the channel can be narrowed while still meeting flood control objectives. This may require increasing levee or floodwall heights. In the former case, additional right of way may be needed to accommodate the wider levee footprint. Additionally, the evaluation should consider whether the problem could be remedied by slope alteration or modification to downstream structures that constrict the flow and cause backwater into the area of concern.

3.1.5.3 Gradient Alteration

The current channel gradient varies dramatically from near 3 percent at the upstream end to below 0.5 percent at the downstream end. Though there is a strong trend for decreasing gradient in the downstream direction, there are localized areas where the gradient changes abruptly. This is partially due to the wide range of channel configurations currently found in the project area. At the current level of design, the proposed channel sections have been superimposed on the existing channel gradient. In the next level of design, the profile needs to be refined considering minimizing changes in sediment transport capacity that result from local variations in the gradient. Additionally, this exercise will likely have benefits to the providing the most efficient flood control design.

3.1.5.4 Structures

Numerous structures are located throughout the project area and upstream reaches, including 13 stream crossings and several energy dissipators. Some of the bridges create constrictions that result in backwater and induce sediment deposition upstream. It is believed that the modifications to these bridges to provide passage of floods should solve these problems, but sediment transport modeling should still be performed to substantiate this. Because of the channel alterations, the energy dissipation structures will be removed by others and will not be a factor under the with-project condition.

3.1.6 Provide Opportunities for Environmental Enhancement

Though the purpose of the project is flood control, environmental features have been identified as important aspects to local stakeholders. Therefore existing areas with higher environmental values should be preserved and in other areas it may be possible to increase the environmental values over current conditions. Channel morphology and sediment transport aspects of the channel design can play a role in preventing loss of existing high environmental value areas and to enchaining the environmental values in other areas. For example, the Greenbelt Reach upstream of the project area has environmental values that are not found in the project area. However, this is the reach that would likely be most susceptible to increase in changes in sediment supply. In other portions of the channel, creation of benches to provide at least limited floodplain can provide environmental enhancement. Also, the design of the channel influences the aquatic habitat. The most significant opportunities to provide environmental enhancement that relate to sediment transport, geomorphology and channel stability are listed below:

- Create a channel with an alluvial bed
- Utilize vegetation to the extent possible to provide bank stability
- Develop a main channel that conveys flows that are on the order of the 50% chance exceedance event
- Provide an area adjacent to the main channel that serves as a floodplain
- Promote growth of vegetation on the floodplain
- Avoid overly wide channels that spread flows very shallow

These opportunities have all been taken advantage of in alternatives 4B, with the extent of vegetation dependent on the further selection of vegetation types for the benches. Alternative 2B incorporates an alluvial channel and may incorporate some vegetation, but does not address the other environmental opportunities listed.

3.2 Qualitative Evaluation of Sediment Transport

This section presents a qualitative assessment of changes in sediment transport conditions and the potential changes in channel response based on comparisons of with- and without-project hydraulic conditions. The two hydraulic parameters chosen to perform the evaluation are velocity and shear stress. Sediment transport is sensitive to these parameters with sediment transport capacity typically increasing with velocity raised to a power of 3 to 5. Shear stress determines the sizes of bed material that can be mobilized. The qualitative evaluation of sediment transport is presented for the preliminary array of alternatives and for the final array of alternatives.

3.2.1 Preliminary Array of Alternatives

As described in Section 2.1 and Chapter 4 of *Part I: Hydraulic Analysis of Alternatives* of this engineering appendix, HEC-RAS models were developed to model the without-project condition and preliminary array of alternatives. To assess potential changes in sediment transport conditions within the project area, velocity and shear stress values from the original GRR methodology (see Section 2.1 of *Part I: Hydraulic Analysis of Alternatives* of this engineering appendix) HEC-RAS models were compared from reach to reach along the channel. The plots were reviewed for without-project baseline and the with-project alternatives. The velocity plots are presented in Figure 3-1 and Figure 3-2 for the 50% chance exceedance events and Figure 3-5 and Figure 3-6 for the 1% chance exceedance events. Similar shear stress versus project station plots are provided in Figure 3-3 and Figure 3-4 for the 50% chance exceedance events and Figure 3-8 for the 1% chance exceedance events. All figures have been separated into two plots (part 1 containing baseline, Alternatives 2A, 3A, and 3B and part 2 containing baseline, Alternative 4B and Alternative 5), plotted at the same scale, to facilitate easy comparison with baseline conditions. Results have been smoothed with running average values over two cross sections upstream and downstream of each station. Sections 2.1.2 and 4.3 of Part I: Hydraulic Analysis of Alternatives of this engineering appendix contains more comprehensive results for the original GRR methodology without-project and preliminary alternatives.

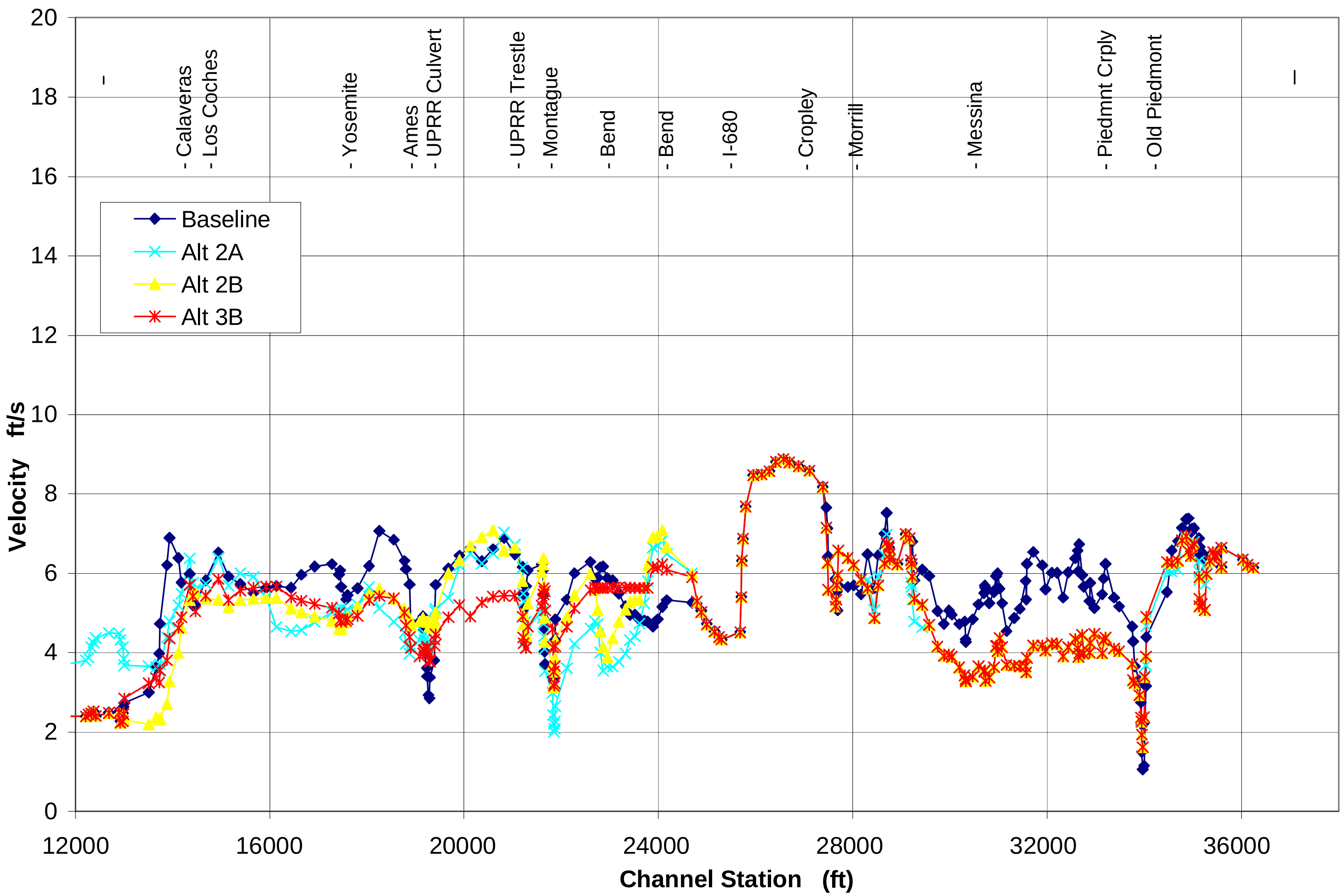

Figure 3-1 (Part 1 of 2) – Main Channel Velocity for Without- and With-Project Conditions, 50% Chance Exceedance Event

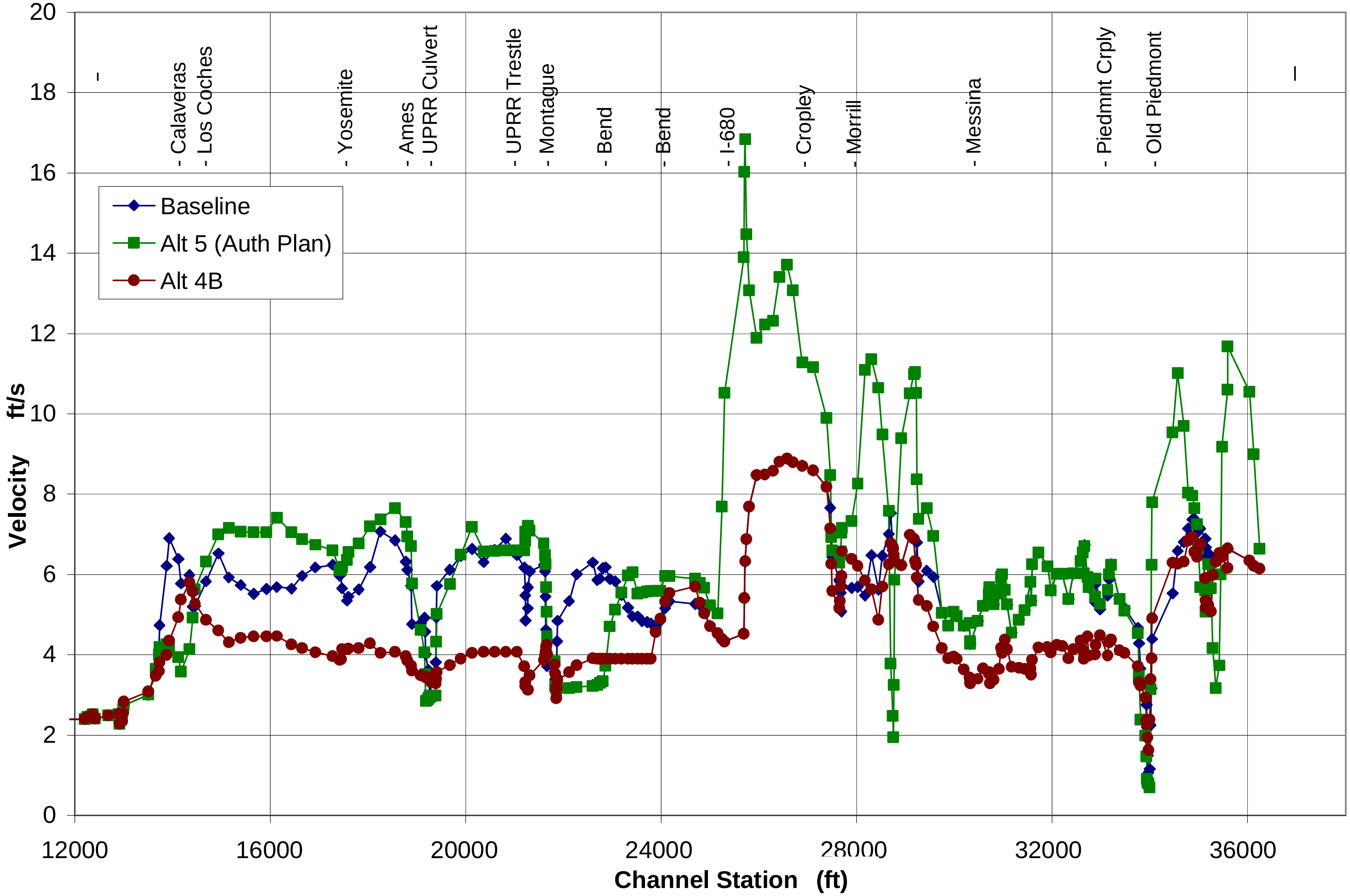

Figure 3-2 (Part 2 of 2) – Main Channel Velocity for Without- and With-Project Conditions, 50% Chance Exceedance Event

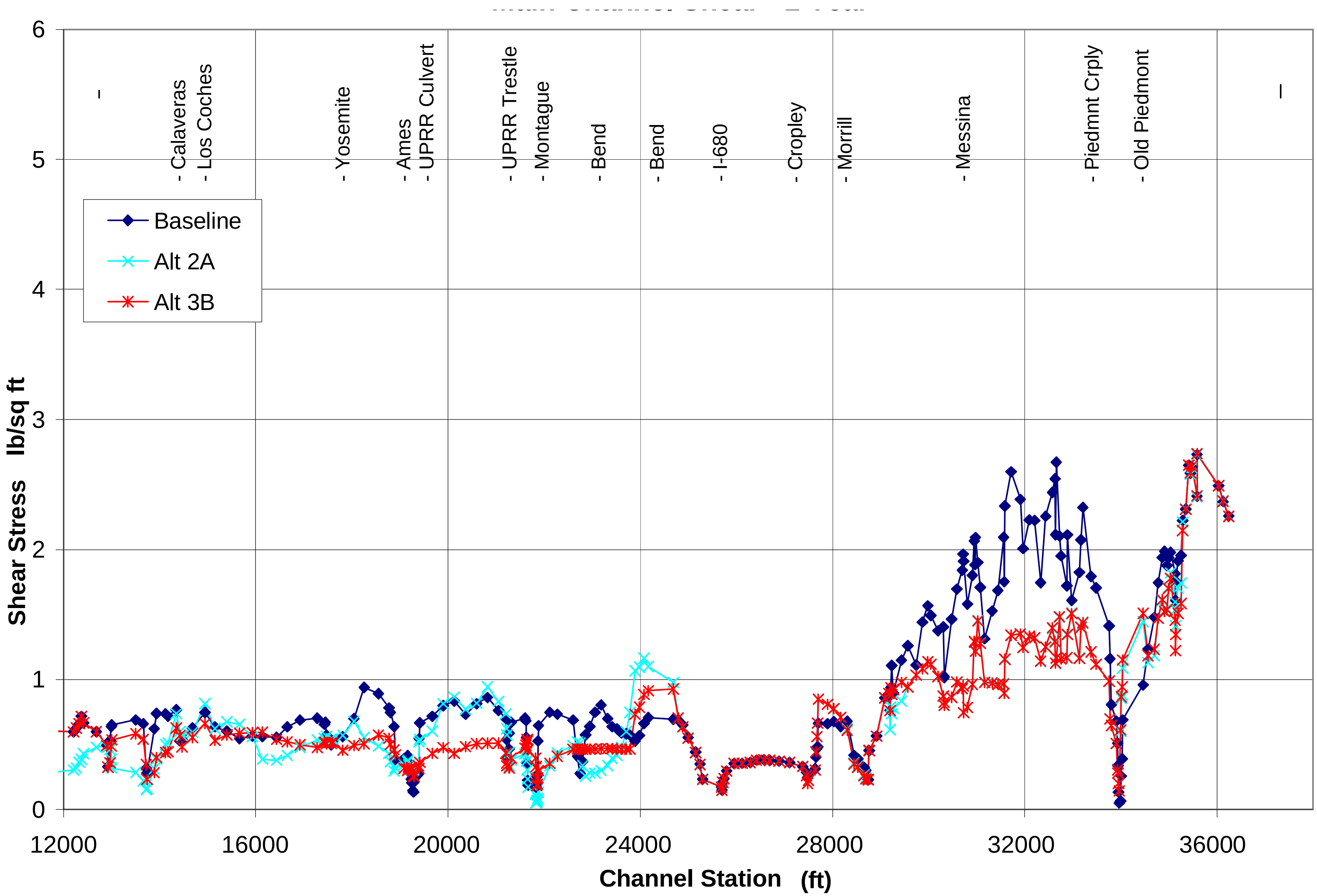

Figure 3-3 (Part 1 of 2) – Main Channel Shear Stress for Without- and With-Project Conditions, 50% Chance Exceedance Event

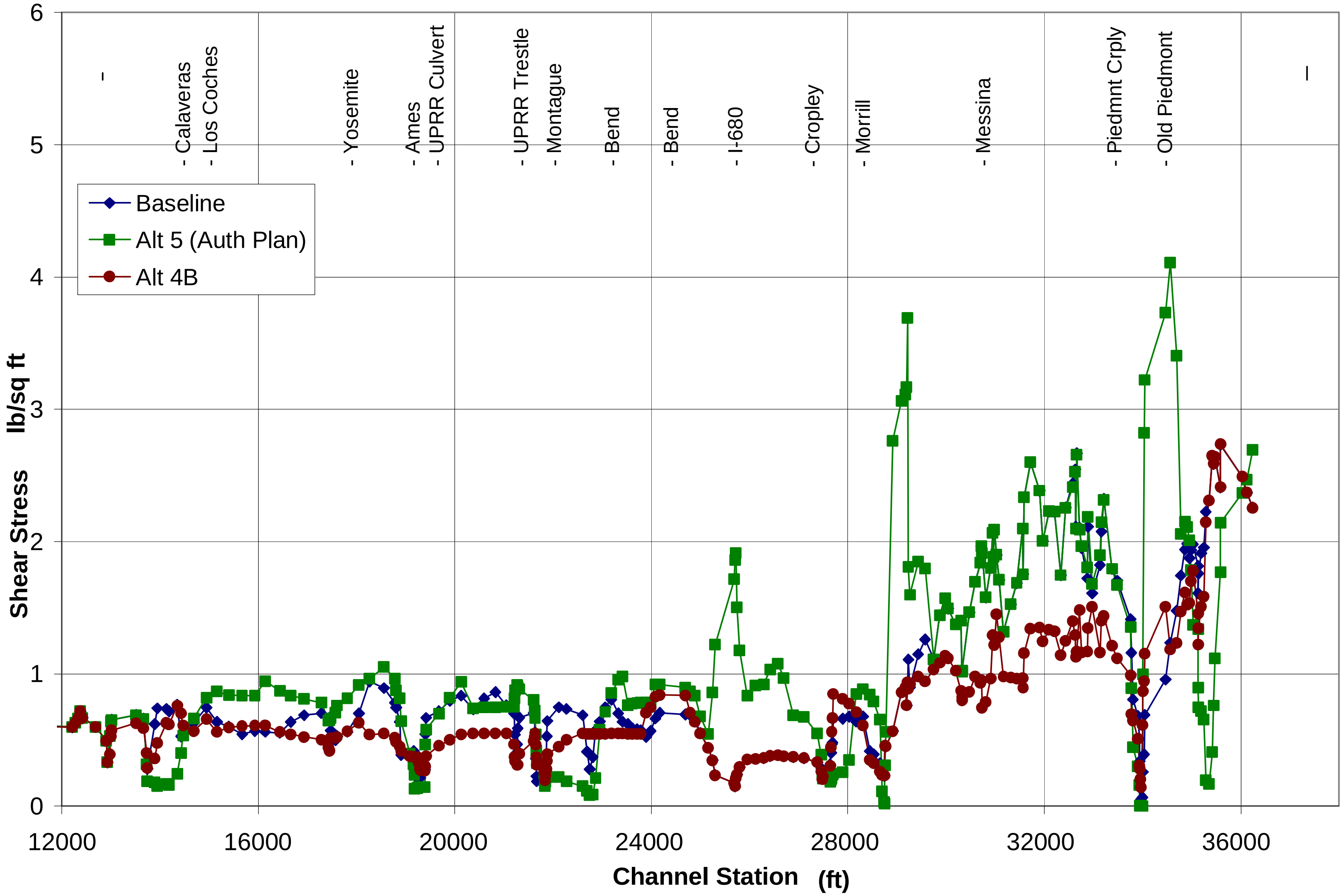

Figure 3-4 (Part 2 of 2) – Main Channel Shear Stress for Without- and With-Project Conditions, 50% Chance Exceedance Event

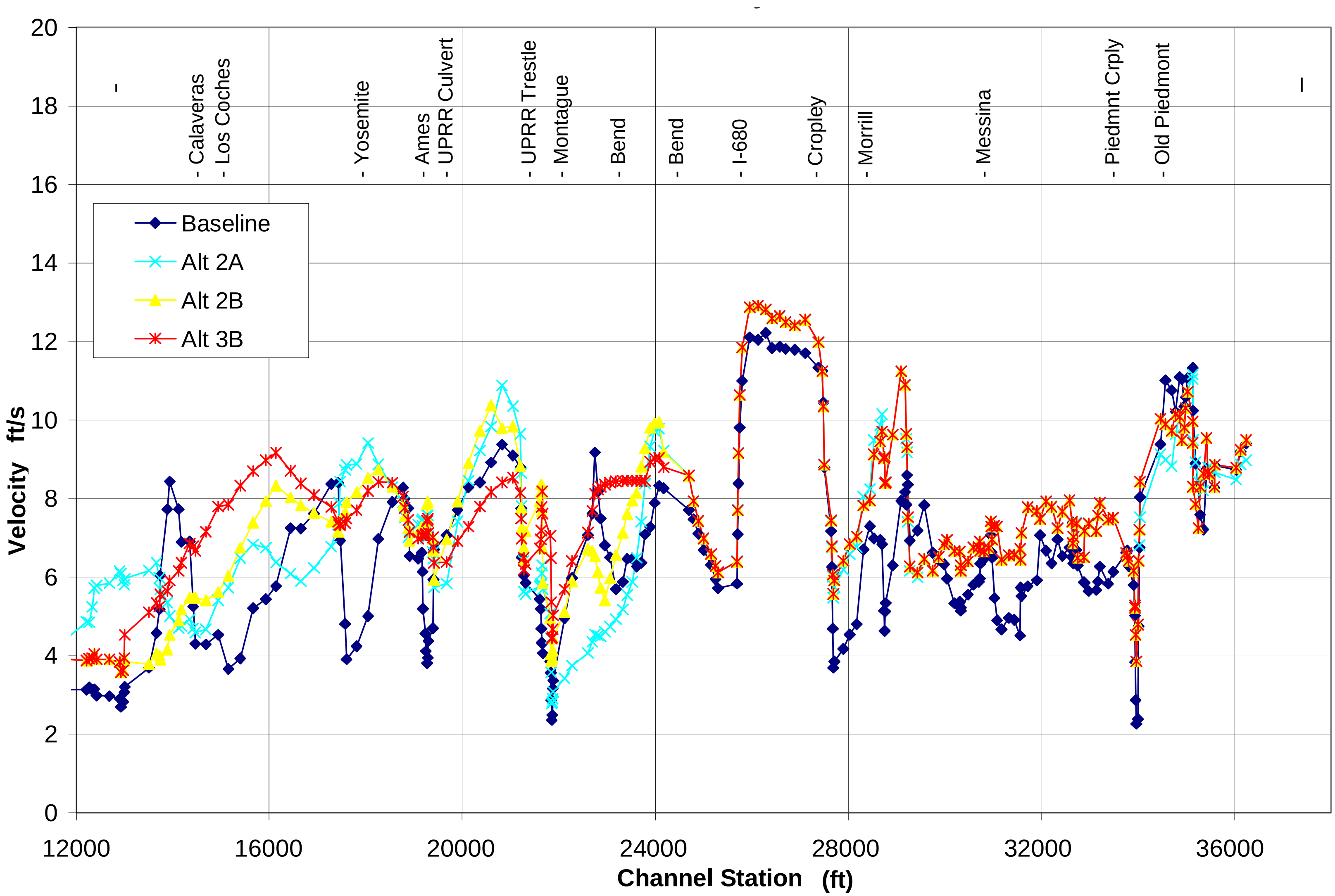

Figure 3-5 (Part 1 of 2) – Main Channel Velocity for Without- and With-Project Conditions, 1% Chance Exceedance Event

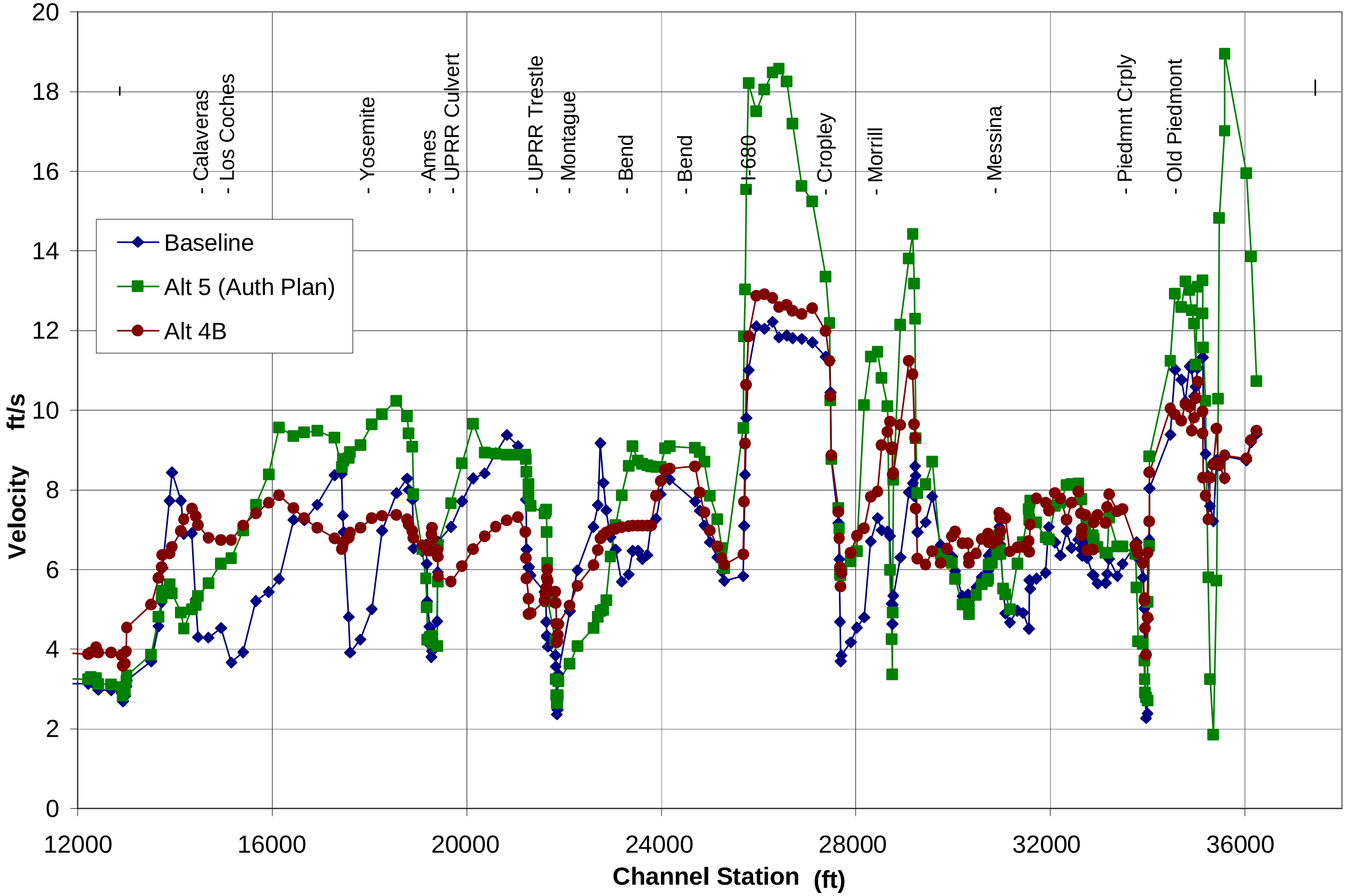

Figure 3-6 (Part 2 of 2) – Main Channel Velocity for Without- and With-Project Conditions, 1% Chance Exceedance Event

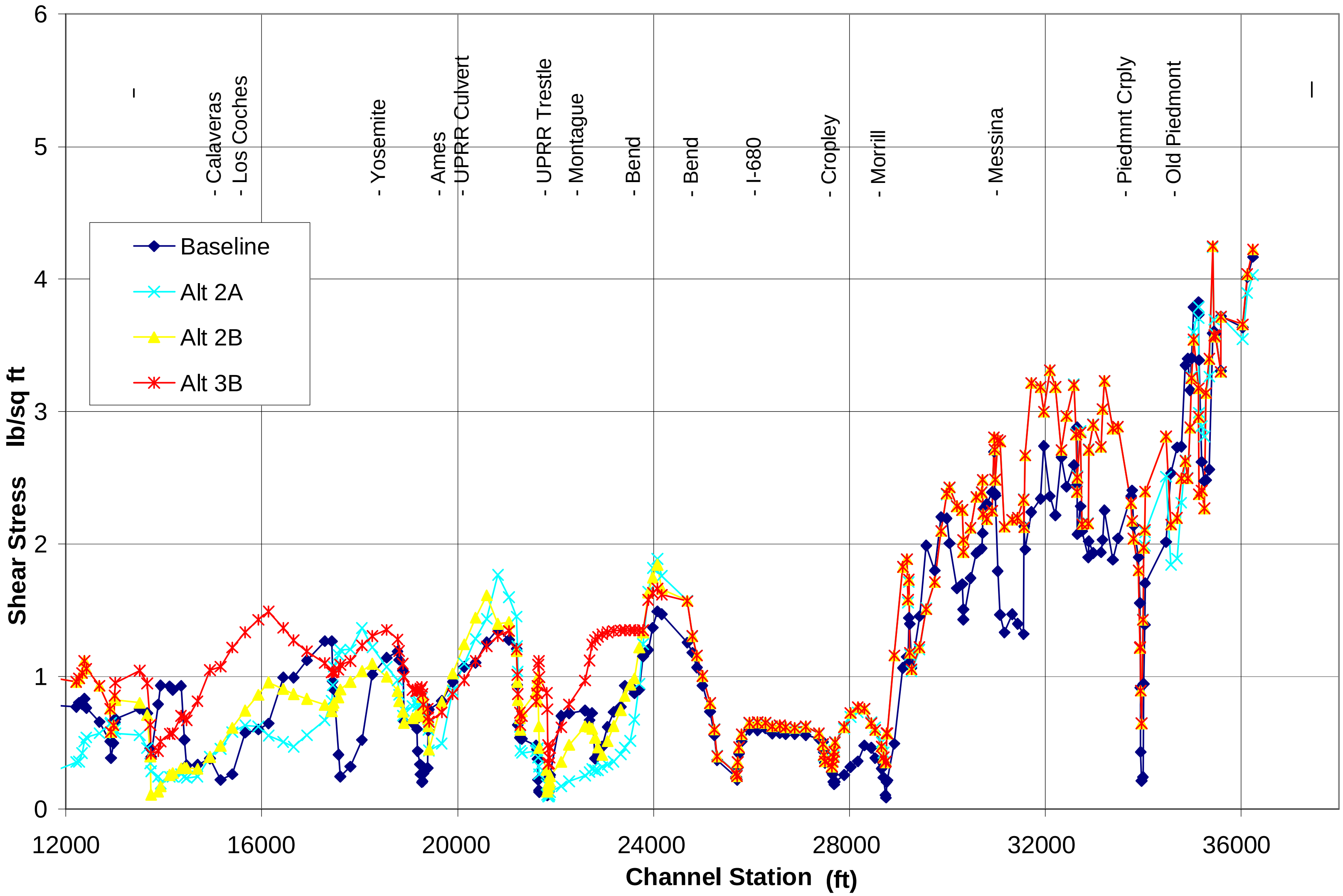

Figure 3-7 (Page 1 of 2) – Main Channel Shear Stress for Without- and With-Project Conditions, 1% Chance Exceedance Event

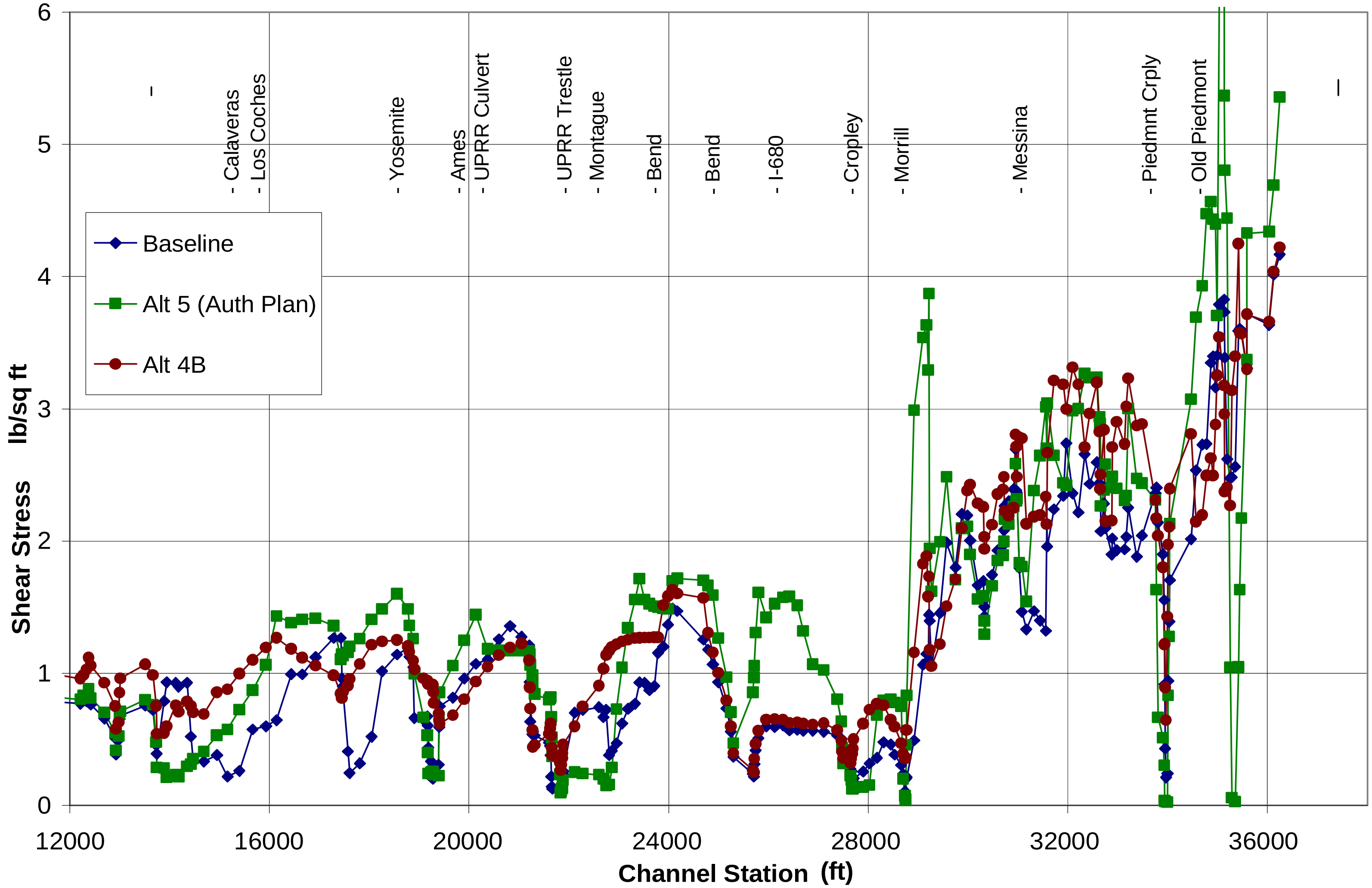

Figure 3-8 (Page 2 of 2) – Main Channel Shear Stress for Without- and With-Project Conditions, 1% Chance Exceedance Event

The values in both sets of plots are for the main channel since this is the portion of the flow that is responsible for nearly all the bed material load transport and it is the bed material load transport that determines the aggradation and degradation characteristics within the project area. Additionally, it is the sand and larger material that has been removed from the channel and sediment basin by past maintenance activities. The larger variation in shear stresses and velocities in the alternatives are related to the in-line detention basins, with backwater conditions behind and weir flow over the crest.

3.2.1.1 Comparison of 50% Chance Exceedance Event

The 50% chance exceedance event was used in the comparison because this event is considered to be approximately the channel forming flow, i.e. most representatives of typical conditions that determine the behavior of the channel over the long term.

<u>Velocity</u>

There is a general trend in reduction of the 50% chance exceedance event velocity for the with-project condition in the Calaveras Boulevard to Montague Expressway reach. Starting from the downstream end of the project, in the reach extending 500 feet upstream of Calaveras Boulevard, the velocities for all alternatives decrease by between 2 and 7 feet per second. The without-project velocity spikes at station 141+21 at 11 feet per second while the with-project velocities range from 3 to 7 feet per second. The largest decrease in this area is with Alternatives 2B and Alternative 5. For the rest of the distance up to Montague Expressway, the velocities for Alternatives 2A, 2B and 3B are similar to without-project condition, except where the velocity spikes (to almost 10 feet per second) downstream on the UPRR culvert; these higher values are eliminated for these with-project alternatives. A high velocity spike of nearly 9 feet per second is introduced in Alternative 2B immediately upstream of the UPRR culvert. The velocities for Alternative 4B are generally lower than the without-project condition in this reach, and the velocities for Alternative 5 are slightly higher than the without-project condition.

Upstream of I-680 to Morrill Avenue, the with-project conditions are extremely similar to the without for all alternatives except Alternative 5. Alternative 5 contains similar velocities to the without-project condition in some of this reach, but varies in particular in the vicinity of bridges due to differing conveyance capacity of the bridges and culverts in this alternative.

Upstream of Morrill Avenue to the upper extent of the Greenbelt area, the velocities of the without-project condition are generally higher than Alternative 2A, 2B, 3B and 4B, oscillating between roughly 3 and 8 feet per second. Many of the spikes are approximately 50 % higher than the values for these Alternatives (8 feet per second compared at 5 to 6 feet per second). Conversely, Alternative 5 has very similar velocities to the without-project condition in this reach, with the exception of two very high velocity spikes of 16 and 17 feet per second at stations 344+67 and 355+86 respectively.

Shear Stress

The comparison of shear stress for the 50% chance exceedance event show similar trends to the velocity comparison described previously. In the vicinity of Calaveras Boulevard, the shear stresses drop by 0.5 to 1 lbs/ft^2 for all with-project Alternatives. In the reach extending from Calaveras Boulevard up to I-680, shear stresses for all Alternatives are on average slightly lower than the without-project condition. Between I-680 and Morrill Avenue shear stresses of Alternatives 2A, 2B, 3B and 4B are identical to the with-project condition, typically 0.5 to 1 lbs/ft^2. From Morrill Avenue to the project upstream limit, shear stresses of the without-project condition oscillate considerably between 1 and 4 lbs/ft^2. Values for Alternatives 2A, 2B, 3B and 4B oscillate, generally between 1 and 2.5 lbs/ft^2. Alternative 5 differs significantly from the other with-project alternatives, due to the presence of in-line detention basins and the differing conveyance capacities of the bridges and culverts.

3.2.1.2 Comparison of 1% Chance Exceedance Event

The 1% chance exceedance event was used in the comparison because it is a large event that is typically utilized to represent the most severe conditions that the project is likely to experience during its design life. Though the 50% chance exceedance event indicates the general behavior of the project over a long period, the response during the 1% chance exceedance event can cause damages that can require significant maintenance or destroy project features.

Velocity

For the 1% chance exceedance event velocity, the velocity changes in the area of Calaveras Boulevard are more significant than for the 50% chance exceedance event. From 1,000 feet downstream to Calaveras Boulevard, they increase by about 1 foot per second for all with-project conditions, Alternative 2A showing a greater increase of up to 3 feet per second. At station 141+21, the without-project velocity spikes to 12 feet per second, whereas the velocities for the with-project alternatives are lower ranging from 5 and 8 feet per second. From upstream of Calaveras Boulevard to I-680, there is no clear trend between the with- and without-project conditions. Though the velocities are not the same, they all vary widely from about 4 feet per second to 12 feet per second, with similar averages through the reach but with significant differences at individual locations. Generally, velocities for the without-project condition spike and fall to a greater degree than for the with-project alternatives. Between the UPRR culvert and Trestle, Alternative 2A has two spikes over 12 feet per second, whereas Alternatives 2B, 3B, 4B and 5 are consistently between 8 to 10 feet per second. The baseline condition varies from 6 to 10 feet per second in this reach.

From Montague Expressway and upstream for 1,000 feet, the velocities drop by several feet per second for all alternatives, with Alternative 2A having the largest drop. The with-project conditions in this segment are the lowest in the entire project area, generally dropping to a maximum of 3 feet per second. Whereas the without-project condition has velocities of 3 to 4 feet per second only in the area of the Montague Expressway bridge, the with-project conditions velocities remain in the 3 to 4 feet per second range for approximately 1,000 feet upstream. This is not desirable, since the area already experiences sediment deposition.

Further upstream between stations 260+00 and 300+00 the velocities for Alternatives 2A, 2B, 3B and 4B are extremely similar to the without-project condition. In the vicinity of the I-680 crossing, velocities under all project scenarios drop to 5 feet per second, but upstream of this the velocities in all cases increase to 12 to 13 feet per second. Alternative 5 shows much larger velocity spikes, over 20 feet per second, in this reach. Between Old Piedmont Road and I-680 to the upstream project limit, velocities oscillate to a greater degree for all Alternatives and the without-project condition, with values ranging between 5 and 10 feet per second. Again, Alternative 5 is the exception with spikes near to the project upstream limit of over 25 feet per second.

Shear Stress

The comparison of shear stress for the 1% chance exceedance event show similar trends to the velocity comparison. The with- and without-project conditions shear stresses overall for the 1% chance exceedance event indicate a drop of around 1 lbs/ft^2 for the with-project conditions. Overall the drop is least for Alt 3B and most substantial for Alt 2B. Alternative 2A has a high spike in shear stress at two locations between the UPRR culvert and trestle greater than 2 lbs/ft^2. Similar to velocity, there is a significant drop in shear stress in the vicinity and upstream of Montague Expressway. Values drop below 0.2 lbs/ft^2 for all alternatives. Between station 240+00 and 280+00 the shear stresses for all Alternatives except Alternative 5 are identical to the without-project condition. Between Old Piedmont Road and I-680, the with- and without-project shear stresses oscillate considerably between 1 and 6 lbs/ft^2. This is true mostly for Alternative 5, except for two large spikes of 11 and 17 lbs/ft^2.

3.2.2 Final Array of Alternatives

As described in Section 2.2 and Chapter 5 of *Part I: Hydraulic Analysis of Alternatives* of this engineering appendix, unsteady HEC-RAS models were developed as part of this study to model the without-project and final array of project alternatives. To assess potential changes in sediment transport conditions within the project area, velocity and shear stress values from the revised GRR methodology (see Section 2.2 of *Part I: Hydraulic Analysis of Alternatives* of this engineering appendix) HEC-RAS models were compared from reach to reach along the channel. During the analysis of the preliminary array of alternatives it was found that the portion of the project between Old Piedmont Road and I-680 was not justified and those portions of the project were removed from the final alternatives. Therefore, the following figures show only the downstream of I-680 results. The trends apparent in the plots were reviewed for without-project and with-project alternatives. The velocity plots are presented along the project station line in Figure 3-9 and Figure 3-11 for the 50% and 1% chance exceedance events, respectively. Similar plots are provided in Figure 3-10 and Figure 3-12 for shear stress. Results have been smoothed with running average values over two cross sections upstream and downstream of each station. Sections 2.2.2 and 5.4 of *Part I: Hydraulic Analysis of Alternatives* of this engineering appendix contains more comprehensive results for the revised GRR methodology without-project and final array of alternatives.

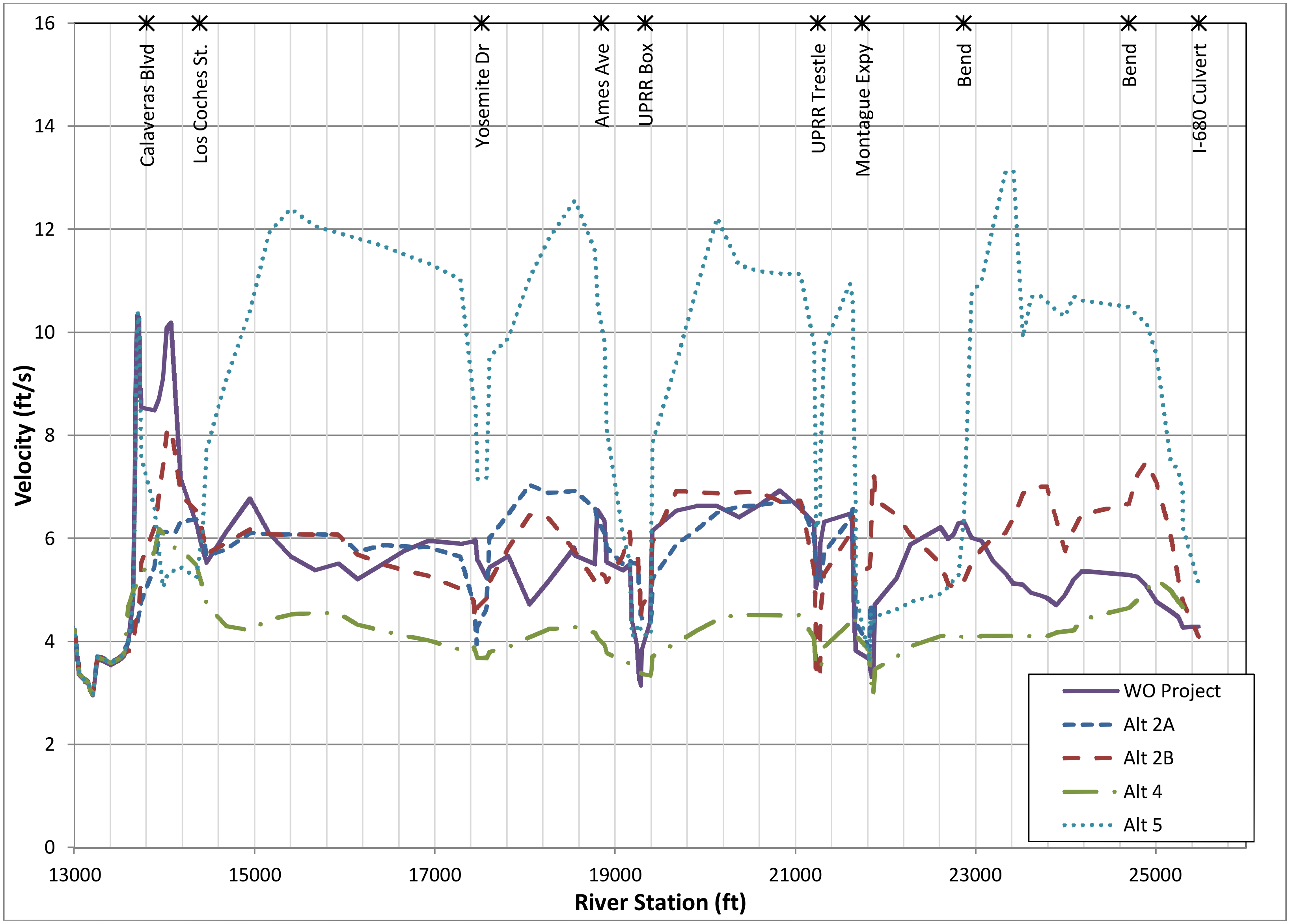

Figure 3-9 Main Channel Velocity Comparison of Without- and With-Project Conditions, 50% chance exceedance Event

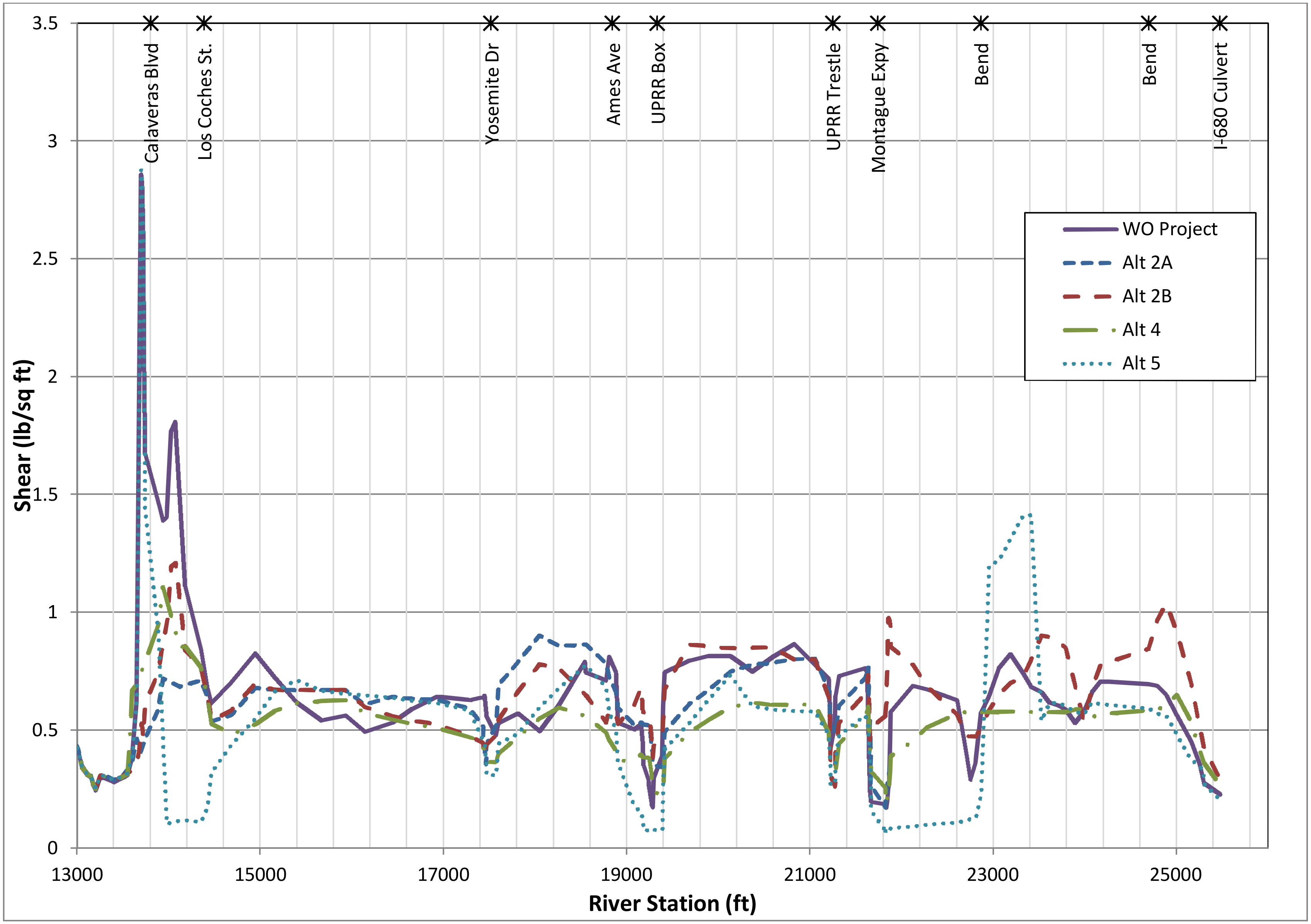

Figure 3-10 Main Channel Shear Stress Comparison of Without- and With-Project Conditions, 50% chance exceedance Event

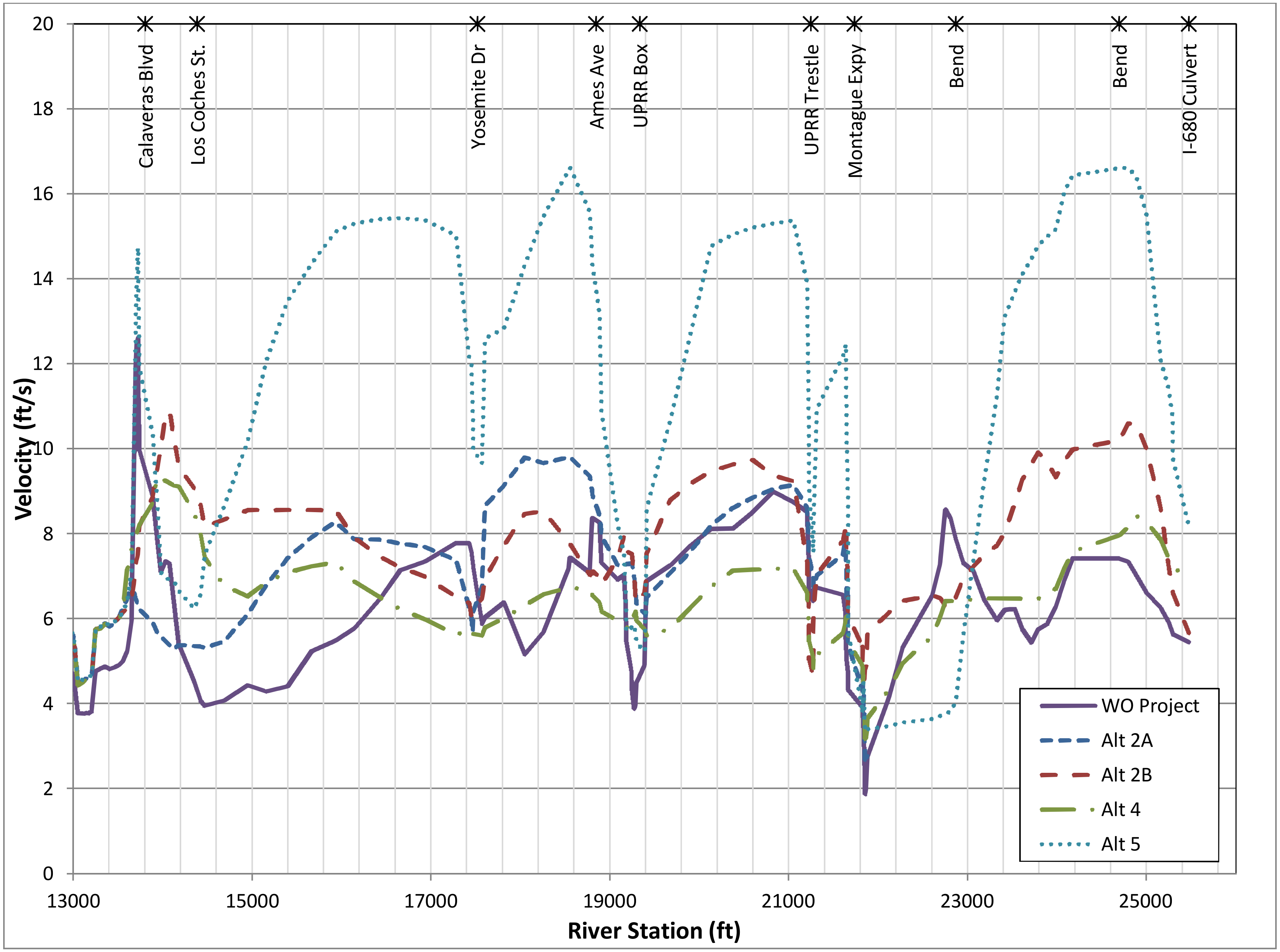

Figure 3-11 Main Channel Velocity Comparison of Without- and With-Project Conditions, 1% chance exceedance Event

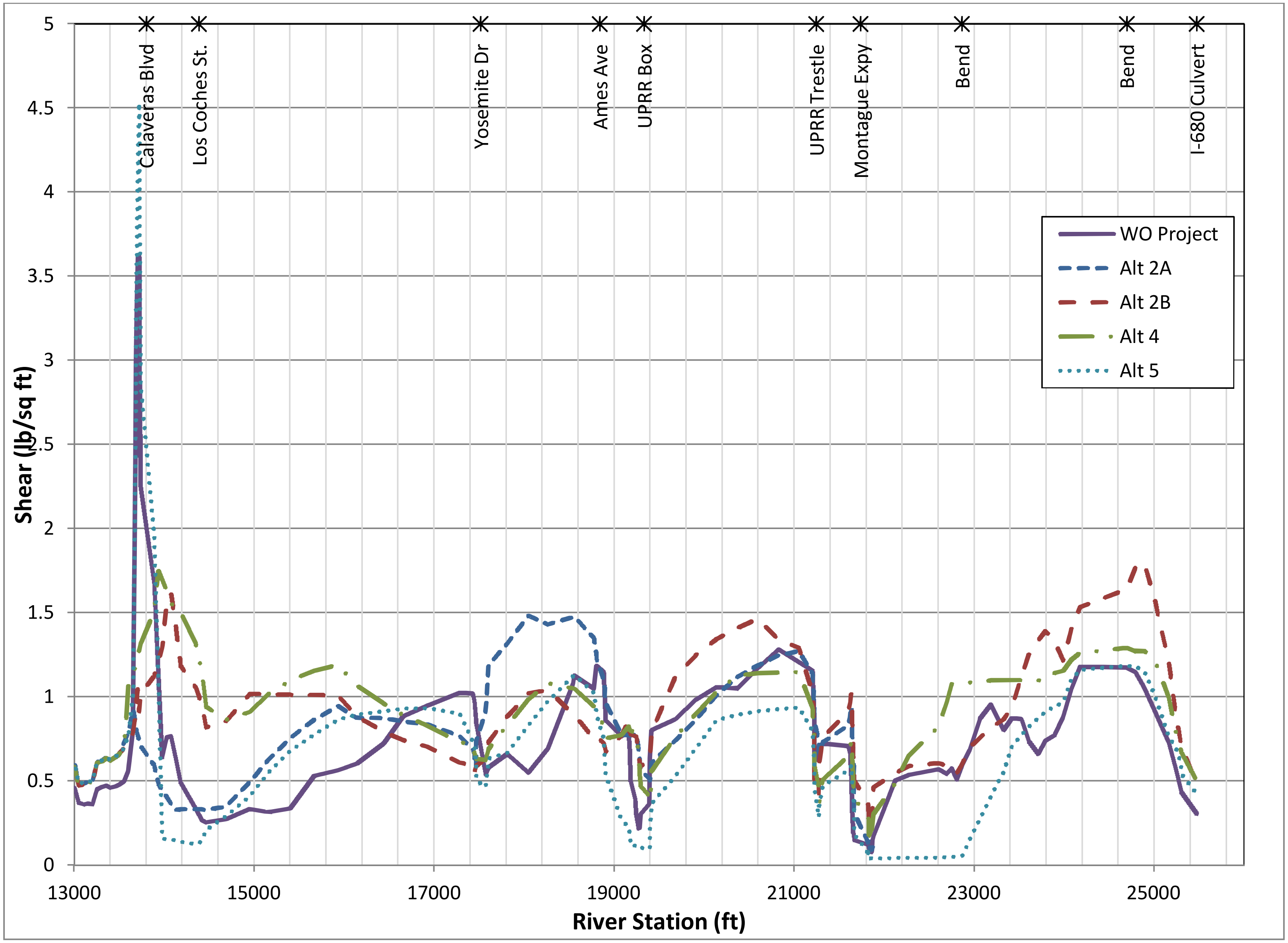

Figure 3-12 Main Channel Shear Stress Comparison of Without- and With-Project Conditions, 1% chance exceedance Event

The values in both sets of plots are for the main channel since this is the portion of the flow that is responsible for nearly all the bed material load transport and it is the bed material load transport that determines the aggradation and degradation characteristics within the Greenbelt and the project area. Additionally, it is the sand and larger material that has been removed from the channel and sediment basin by past maintenance activities.

3.2.2.1 Comparison of 50% Chance Exceedance Event

The 50% chance exceedance event was used in the comparison because this event is considered to be approximately the channel forming flow, i.e., the most representative of typical conditions that determine the behavior of the channel over the long-term.

The general trend in velocity is for Alternatives 2A/d and 2A/b is to approximately follow the without-project velocities with minor reductions in velocities upstream of Montague. Alterative 4/d shows a general reduction of the 50% chance exceedance velocity for the with-project condition relative to the without-project. The decrease is generally on the order of 0.5 up to 2.0 feet per second. In some isolated areas for Alternative 2A/d, 2B/d, and 4/d, particularly where the modification of bridges removed backwater effects, velocities show an increase. Alternative 5 shows a large increase in velocity over the without-project based on the concrete lined channel proposed. The highest running average velocity exhibited under with-project conditions is approximately 7.5 feet per second in Alternative 2B/d.

A comparison of shear stresses for the 50% chance exceedance event shows similar trends to the velocity, with shear stresses for Alternatives 2A/d, 2B/d, and 4/d on average equal to or slightly lower than the without-project condition. In a few areas, specifically above Montague Blvd and downstream of Yosemite Ave., the alternative shear stress is higher than the without project conditions. Shear stress for Alternative 5 is generally lower than the without-project conditions with the exception of two locations, one upstream of Montague Blvd. and one downstream of Yosemite Ave., that are higher than the without project condition.

3.2.2.2 Comparison of 1% Chance Exceedance Event

The 1% chance exceedance event was used in the comparison because it is a large event that is typically utilized to represent the most severe conditions that the project is likely to experience during its design life. Though the 50% chance exceedance event indicates the general behavior of the project over a long period, the response during the 1% chance exceedance event can cause damages that can require significant maintenance or destroy project features. Under existing conditions, the 1% chance exceedance discharge breaks out of the channel in several locations. The with-project alternatives contain a larger discharge and result in velocity and shear stress increases downstream of breakout locations. The increases in velocity are most pronounced in the reaches where the right-of-way is constrained. The maximum running average velocities exhibited under with-project conditions are approximately 16.5 feet per second in Alternative 5.

A comparison of shear stresses for the 1% chance exceedance event shows similar trends to the velocity comparison. The maximum running average shear stress under with-project conditions is approximately 1.8 lbs/sq ft for both Alternatives 2B/d and 4/d.

3.3 Quantitative Sediment Transport Analysis of the Final Array of Alternatives

A quantitative sediment transport analysis was conducted for the final array of alternatives. The purpose of the analysis was to develop an estimate of the potential O&M sediment removal quantities for the Final Array of Alternatives assuming existing conditions between Old Piedmont Road and I-680. In addition, an analysis was conducted assuming the SCVWD Bypass Alternative was in place between Old Piedmont Road and I-680 for Alternatives 2B/d and 4/d.

3.3.1 Methodology

This section presents the methodology used to conduct the sediment transport analysis. Due to differing levels of information being available between Old Piedmont Road and I-680 for the existing conditions and SCVWD Bypass alternatives, different methodologies were used for each analysis.

3.3.1.1 Existing Conditions between Old Piedmont Road and I-680 Methodology

A spreadsheet analysis of the sediment transport capacity through the study area was conducted to determine the potential O&M requirements for the final array of alternatives. The study area was divided into four reaches based on the reaches used to report sediment removal maintenance provided by SCVWD (as discussed in Section 3.1.4). Additionally, *Upstream of the Piedmont-Cropley Culvert* and the *Greenbelt between the Piedmont-Cropley Culvert and Morrill Avenue* were added as supply reaches, since these reaches are a source of sediment supply to the downstream reaches. The transport reaches used are listed in Table 3-1.

Table 3-1 Analysis Reaches

Reach	Reach Type
Upstream of the Piedmont-Cropley Culvert	Supply
Greenbelt between Piedmont-Cropley Culvert and Morrill Ave	Supply
Morrill Ave to I-680	Transport
I-680 to Montague Expressway	Transport
Montague Express to Calaveras Blvd	Transport
Downstream of Calaveras Blvd	Transport

The Yang sediment transport equation was used to estimate the sediment transport through each reach. The Yang sediment transport equation was chosen based on the research conducted by Brett Jordan on Berryessa Creek for his dissertation in 2009 (Jordan, 2009). Jordan concluded that the Yang equation best represented Berryessa Creek based on an analysis of potential sediment transport equations. The Yang equation has two variations based on whether the transport of sand and gravel is being estimated. The Yang equation estimates the sediment transport rate based on a representative diameter and reach-averaged hydraulics.

Sediment gradation curves were obtained from sediment sampling conducted for the Northwest Hydraulic Consultants' *Upper Berryessa Creek Existing Conditions Sediment Transport Assessment* (NHC, 2003). A number of samples were collected along each reach during different times of the year. For the purposes of this analysis samples taken during the winter season were used since the high flows in Berryessa Creek occur primarily during the winter rainy season. For the purpose of this analysis, the sediment gradation curves were divided into ten sediment size classes with a representative diameter assigned to each. The size fraction of each sediment size class was determined for each reach. Table 3-2 lists the minimum, maximum, and representative diameters for each of the sediment sizes classes used. Table 3-3 lists the fraction of the total for each sediment size class for each reach.

Table 3-2 Sediment Size Classes

Grain Size Interval	Min Diameter	Max Diameter	Representative Diameter
Fine/Very Fine Sand	0	0.25	0.125
Medium Sand	0.25	0.5	0.35
Course Sand	0.5	1	0.71
Very Coarse Sand	1	2	1.4
Very Fine Gravel	2	4	2.8
Fine Gravel	4	8	5.7
Medium Gravel	8	16	11.3
Course Gravel	16	32	22.6
Very Course Gravel	32	64	45.8
Small Cobble	64	128	91.6
Total			

Table 3-3 Sediment Class Size Distribution by Reach

Grain Size Interval	Sediment Class Size Distribution					
	Upstream of the Piedmont-Cropley Culvert	**Greenbelt from Piedmont-Cropley Culvert to Morrill Ave**	**Morrill Ave to I-680**	**I-680 to Montague Expressway**	**Montague Express to Calaveras Blvd**	**Downstream of Calaveras Blvd**
Fine/Very Fine Sand	6%	5%	6%	4%	3%	4%
Medium Sand	6%	6%	7%	7%	6%	10%
Course Sand	4%	5%	6%	7%	6%	10%
Very Coarse Sand	7%	7%	9%	14%	14%	13%
Very Fine Gravel	7%	12%	13%	18%	16%	15%
Fine Gravel	10%	17%	17%	16%	20%	18%
Medium Gravel	12%	20%	17%	19%	22%	18%
Course Gravel	21%	18%	16%	11%	11%	9%
Very Course Gravel	8%	6%	7%	4%	2%	3%
Small Cobble	19%	4%	2%	0%	0%	0%
Total	**100%**	**100%**	**100%**	**100%**	**100%**	**100%**

The average hydraulics for the 50% to 0.2% chance exceedance events were developed for each reach using the results of the FLO-2D and HEC-RAS modeling discussed in *Part I: Hydraulic Analysis of Alternatives* and *Part II: Floodplain Development of Alternatives*. Since the bulk of the average annual sediment transport is conveyed proportionally by smaller, more frequent events, a 67% chance exceedance event was developed. The 67% chance exceedance event was developed by plotting the inflows to the FLO-2D and HEC-RAS models and estimating the 67% chance exceedance event inflows. The ratio of the 67% to the 50% chance exceedance inflows was then computed and applied to the FLO-2D and HEC-RAS 50% chance exceedance inflows used to develop the hydraulics for the 67% chance exceedance event.

The reach-averaged hydraulics were used in conjunction with the sediment size class data to calculate the sediment transport for each sediment size class for each event. The total sediment transport rates for each event were developed by combining the calculated transport rates for each sediment class size based on based on the fraction of the total sediment gradation each class represented. Finally, the sediment transport rates for each event were probability-weighted to develop the average annual sediment transport rate for each reach.

The potential deposition in each reach was determined by subtracting the sediment transport through the reach from the transport rate of the reach upstream. A positive result indicated a reduction in the sediment transport capacity through the reach resulting in deposition. A negative result indicated an increase in sediment transport capacity through the reach resulting in pass-through conditions and potential erosion in unarmored section of channel.

Deposition in the sediment basin below the Piedmont-Cropley culvert was developed assuming that 100% of the gravels from the upstream reach were captured in the sediment basin. The amount of sand captured in the sediment basin was calculated based on the assumption that captured sediment matrix was composed of 75% gravel and 25% sand, with the sand filling voids in the gravel.

The initial without-project alternative results were compared to the average annual sediment removal based on maintenance records (see Section 3.1.4) to determine how well the spreadsheet analysis reflected observed deposition trends. As seen in Table 3-4, the initial results did not reflect the observed trend well. To better model the observed deposition calibration coefficients were applied to the sediment transport equations for each of the reaches to better match the observed deposition trends. As seen in Table 3-4 the application of calibration coefficients ranging from 0.98 to 5.31 produced results that matched the observed deposition. The remaining alternatives were analyzed by using the calibrated spreadsheet model and the alternative hydraulics.

Table 3-4 Model Calibration Results

Reach	Average Annual Sediment Deposition (cy)			Calibration Coefficient
	SCVWD Maintenance Records	Initial Results	Calibrated Results	
Upstream Old Piedmont to Piedmont-Cropley Sediment Basin[1]	537	2281	537	0.2355
Piedmont-Cropley Culvert to Morrill Ave (Greenbelt)	0	0	0	2.38
Morrill Ave to I-680	510	-1417	510	0.999
I-680 to Montague Expressway	418	2230	418	4.113
Montague Express to Calaveras Blvd	199	12	199	3.85
Downstream of Calaveras Blvd	5521	557	2180	1

[1]The average annual sediment deposition for this reach is based on the sediment captured in the sediment basin only with no deposition in the reach upstream of the sediment basin.

It should be noted that this methodology was developed based on the limited available hydraulic information. The use of average hydraulics and peaks flows to determine sediment concentrations through reaches represent one point on the sediment rating curve. This approach tends to overestimate the total sediment transport when applied to the entire flow volume from the storm event. A much more intensive modeling approach, beyond the scope of this study, would be required to truly develop the transport based on the sediment transport over the entire range of a storm event. Calibrating the equations to observed deposition trends largely accounts for this effect, thought the results will still be conservative. Therefore, the methodology presented above satisfies the intent to estimate the change in the sediment deposition through the study area.

3.3.1.2 SCVWD Bypass Alternative between Old Piedmont Road and I-680 Methodology

The local sponsor (SCVWD) has proposed a future project between Old Piedmont Road and I-680 consisting of a bypass culvert diverting most of the flood flows around the Greenbelt reach to help alleviate flooding in the Greenbelt reach. The proposed bypass would divert most of the flood flow from Berryessa Creek just upstream of the Piedmont-Cropley culvert, convey the flow down a culvert under Cropley Avenue, and finally discharge the flow at a point near the Cropley Avenue Bridge. The SCVWD bypass alternative is discussed in more detail in Section 5.2.3 in *Part I: Hydraulic Analysis of Alternatives*. The impacts to the sediment maintenance requirements for alternatives 2B/d and Alt 4d were analyzed.

To evaluate the impacts of the SCVWD bypass, the *existing conditions between Old Piedmont Road and I-680* spreadsheet model required modification as detailed hydraulics were not available for the SCVWD bypass alternative. The bypass alters the potential amount of sediment supply from the Greenbelt as well as transporting sediment through the bypass culvert. The transport through the Greenbelt was approximated using the bypass diversion rating curve, the Berryessa Creek flows at the downstream of the Greenbelt, and the existing conditions between Old Piedmont Road and I-680 sediment rating curve for the Greenbelt reach. First the Berryessa Creek peak flows for the *existing conditions between Old Piedmont Road and I-680* at the downstream end of the Greenbelt were determined from the without-project HEC-HMS hydrologic modeling. Then the Berryessa Creek peak flow for the *SCVWD bypass alternatives between Old Piedmont Road and I-680* was developed using the SCVWD bypass HEC-HMS model. A sediment rating curve for the Greenbelt reach was developed using the *existing conditions between Old Piedmont Road and I-680* flows and the calculated sediment transport for each flow event. The sediment rating curve was then used to approximate the sediment transport rate through the greenbelt supply reach based on the Berryessa Creek *with SCVWD bypass alternatives between Old Piedmont Road and I-680* flows at the downstream end of the Greenbelt. .

In addition to altering the sediment transport rate in the greenbelt reach, the SCVWD bypass would also alter the deposition in the sediment basin below the Piedmont-Cropley culvert. To determine the deposition in the sediment basin, the sediment transport through the Piedmont-Cropley culvert was determined for the gravel fraction. A sediment rating curve based on the flow at the culvert for the existing conditions was developed for gravels. The flow through the culvert with the SCVWD bypass in place was then used to approximate the gravel transport through the culvert with the bypass. As for the *existing conditions between Old Piedmont Road and I-680* methodology, it was assumed that 100% of the gravel transported through the culvert would be captured in the basin and that the captured sediment matrix would consist of 75% gravel and 25% sands. Since the invert of the bypass culvert is one foot above the invert of the Piedmont–Cropley culvert, the gravel bed load is prevented from being conveyed through the bypass culvert. Therefore, the remaining portion of the gravel supply from upstream of the bypass will deposit in the reach. Since no detailed hydraulic results were available for the SCVWD bypass alternative, the location of deposition of this material cannot be determined. The remainder of the sand supply was assumed to be conveyed through the bypass culvert and was added to the sediment supply estimate calculate for the Greenbelt reach.

The deposition estimates for the remaining reaches was then developed using the same procedures as the existing conditions *between Old Piedmont Road and* I-680 methodology. The average hydraulics for the study reaches were developed with the HEC-RAS models run with inflows reflecting the SCVWD bypass in place between Old Piedmont Road and I-680.

3.3.2 Results

The quantitative sediment analysis was conducted for the without-project, alternative 2A/d, 2B/d, and 4/d using hydraulic models developed for previous phases of this study for existing conditions between Old Piedmont Road and I-680. In addition, analyses were conducted for alternatives 2B/d and 4/d assuming the proposed SCVWD bypass alternative was in place between Old Piedmont Road and I-680. The potential deposition for each alternative was developed for each reach.

Table 3-5 lists the estimated average annual sediment transport rates and deposition for the without-project, Alternative 2A/d, 2B/d, and 4/d models using existing conditions between Old Piedmont Road and I-680. As seen in the table, for Alternatives 2A/d and 2B/d there is an increase in sediment transport through the I-680 to Montague and Montague to Calaveras. The increased transport results in a decrease in deposition in the I-680 to Montague reach for alternatives. With a larger amount of sediment being transported through the upstream reach, there in an increase in the amount of deposition in the Montague to Calaveras Boulevard reach for all alternatives over the without-project alternative. Overall, the total amount of sediment deposited in study area for Alternatives 2A/d and 2B/d is nearly equal to that under without-project conditions. For Alternative 4/d there is a marked increase in deposition in the study.

Table 3-5 Average Annual Sediment Transport and Deposition using Existing Conditions between Old Piedmont Road and I-680

Alternative	Reach						
	US of Old Piedmont Rd to Piedmont Cropley Culvert	Piedmont Cropley Sediment Basin	Piedmont-Cropley Culvert to Morrill Ave (Greenbelt)	Morrill Ave to I-680	I-680 to Montague Expressway	Montague Expressway to Calaveras Blvd	DS of Calaveras Blvd
Average Annual Sediment Transport Rate (cy)							
Without-Project	537	0	3318	2809	2391	2192	12
Alt 2A/d	537	0	3318	2809	3166	2161	10
Alt 2B/d	537	0	3318	2809	3836	2202	9
Alt 4/d	537	0	3318	2809	2208	1501	14
Average Annual Deposition (cy)							
Without-Project[1]	-na-	537	-na-	509	418	199	2180
Alt 2A/d	-na-	537	-na-	509	0	648	2151
Alt 2B/d	-na-	537	-na-	509	0	607	2192
Alt 4/d	-na-	537	-na-	509	601	707	1487

-na- not applicable as no deposition was modeled in these reaches since they act as supply reaches to the reaches below them and no deposition was reported in the SCVWD maintenance records.
[1]The without-project deposition values were calibrated to SCVWD sediment removal maintenance records.

Table 3-6 lists the average annual sediment transport rates and deposition results for Alternatives 2B/d and 4/d with the SCVWD Bypass between Old Piedmont Road and I-680. The without-project for existing conditions between Old Piedmont Road and I-680 alternative was included in the table for comparison purposes. As seen in the table there is a significant reduction in the deposition in the sediment basin below the Piedmont-Cropley culvert over existing conditions. This is due to a majority of flood flows being transported through the bypass culvert. The reduction in the flood flows to the Greenbelt reach results in a significant reduction in the sediment supply to the downstream reach. The sediment supply conveyed through the bypass culvert adds to the supply to the downstream reach, but accounts for only a small portion of the reduced Greenbelt sediment supply. As seen in the table, the sediment transport rate for the Morrill to I-680 reach is greater than the combined sediment supply for the Greenbelt and Bypass culvert. Since the sediment transport capacity through the reach is greater than the incoming supply, no deposition is seen in the reach. For both alternatives there is an increase in sediment transport through the I-680 to Montague and Montague to Calaveras reaches over the without-project alternative. The increased transport results in no deposition in the I-680 to Montague reach. Normally, a larger amount of sediment being transported through the upstream reach would result in an increase in the amount of deposition in the Montague to Calaveras Boulevard reach. But since the supply from the Greenbelt reach is limited, the transport capacity of Alternative 2B/d can transport the entire supply to the downstream reach with no deposition and Alternative 4/d showing a small amount of deposition.

Table 3-6 Average Annual Sediment Transport and Deposition for the SCVED Bypass between Old Piedmont Road and I-680

Alternative	Reach								
	US of Old Piedmont Rd to Piedmont Cropley Culvert	**Piedmont Cropley Sediment Basin**	**Bypass Culvert**	**Piedmont-Cropley Culvert to Morrill Ave (Greenbelt)**	**Total Sediment Supply entering the Morrill Ave to I-680 Reach[1]**	**Morrill Ave to I-680**	**I-680 to Montague Expressway**	**Montague Expressway to Calaveras Blvd**	**DS of Calaveras Blvd**
Average Annual Sediment Transport Rate (cy)									
Without-Project for existing conditions between Old Piedmont Road and I-680[2]	537	0	-	2219	2219	1709	1292	1092	38
Alt 2B/d with Bypass	537	0	88	1631	1718	2809	3774	2263	9
Alt 4/d with Bypass	537	0	88	1631	1718	2809	2283	1630	16
Average Annual Deposition (cy)									
Without-Project for existing conditions between Old Piedmont Road and I-680[2]	-na-	537	-	-na-	-na-	509	417	200	1057
Alt 2B/d with Bypass	-na-	450	-na-	-na-	-na-	0[3]	0[3]	0[3]	1709
Alt 4/d with Bypass	-na-	450	-na-	-na-	-na-	0[3]	0[3]	89	1702

1. The sediment supply to Morrill Avenue to I-680 reach is a combination of the transport from the Bypass Culvert and the Greenbelt reaches.
2. The without-project for existing conditions between Old Piedmont Road and I-680 alternative is included for comparison purposes.
3. Since the total supply from the Greenbelt to the reach is less than the transport through the reach zero deposition was recorded and potential erosion was not considered in this analysis.

-na- not applicable: no deposition was modeled in these reaches since they act as supply reaches to the reaches below them and no deposition was reported in the SCVWD maintenance records.

3.4 Conclusions

Several significant conclusions can be drawn from the comparisons of velocities and shear stress between the with- and without-project conditions in reference to the influence of the current alternatives on sediment transport conditions.

Throughout the project area, there are large variations in velocities and shear stresses that can cause localized sedimentation and scour problems. The project design needs to be further refined to reduce the level of these changes. Additionally, the measures used to provide passage of the design event through bridges should be reviewed. In cases in which walls were extended above the bridge deck to contain flows, there may be the creation of significant backwater conditions. The reduced velocity and shear stress may cause an additional potential for additional, localized deposition in an area that in some cases already experiences deposition.

Currently, the project area is a deposition zone and a reduction in velocity will further increase deposition and the need for maintenance. Constructed features should facilitate removal of deposited sediments.

Five sediment basin configurations have been previously evaluated upstream of the project area in order to reduce the downstream maintenance needs. The basin configurations are shown in Table 3-7. The schematic locations are shown in plan view and profile view in Figure 3-13 and Figure 3-14, respectively.

Table 3-7 Summary of Sediment Basin Location Alternatives

Alternative	Name	Description
A	F4A	F4A design concept. Existing basin bed lowered approximately 5 feet with 700-foot length excavated channel at basin outlet.
B	Reduced F4A	F4A design concept with reduced basin lowering (approximately 2.5 feet) and excavated channel length (approximately 350 feet).
C	Downstream Adjacent	Channelization of Berryessa Creek through the existing basin, with construction of a new basin located near the existing basin outlet.
D	Morrill	Channelization of Berryessa Creek through the existing basin, with construction of a new basin downstream of the Greenbelt Reach near Morrill Avenue.
E[1]	Authorized	Construction of a new sediment basin upstream of Old Piedmont Road and modification of existing basin with plunge pool, outlet weir, and 3-foot diameter culvert drain.

Notes: 1. Alternative E is the Proposed Sediment Basin per the 1993 GDM Authorized Project Design. (USACE 1993).

An evaluation of the advantages and disadvantages of each configuration concluded that a combination of the above alternatives would best balance maintenance needs against environmental impacts and hydraulic conveyance capacity. These alternatives are currently under consideration by others, and the design of features within the project reach should be coordinated with the design process of the upstream sediment basin in order to ensure consistent approaches. Recommendations and further details on the sediment basin evaluation are presented in a Technical Memorandum dated January 21, 2009 by Tetra Tech, Inc. (2009a).

Figure 3-13 Plan View of Alternative Sediment Basin Configurations

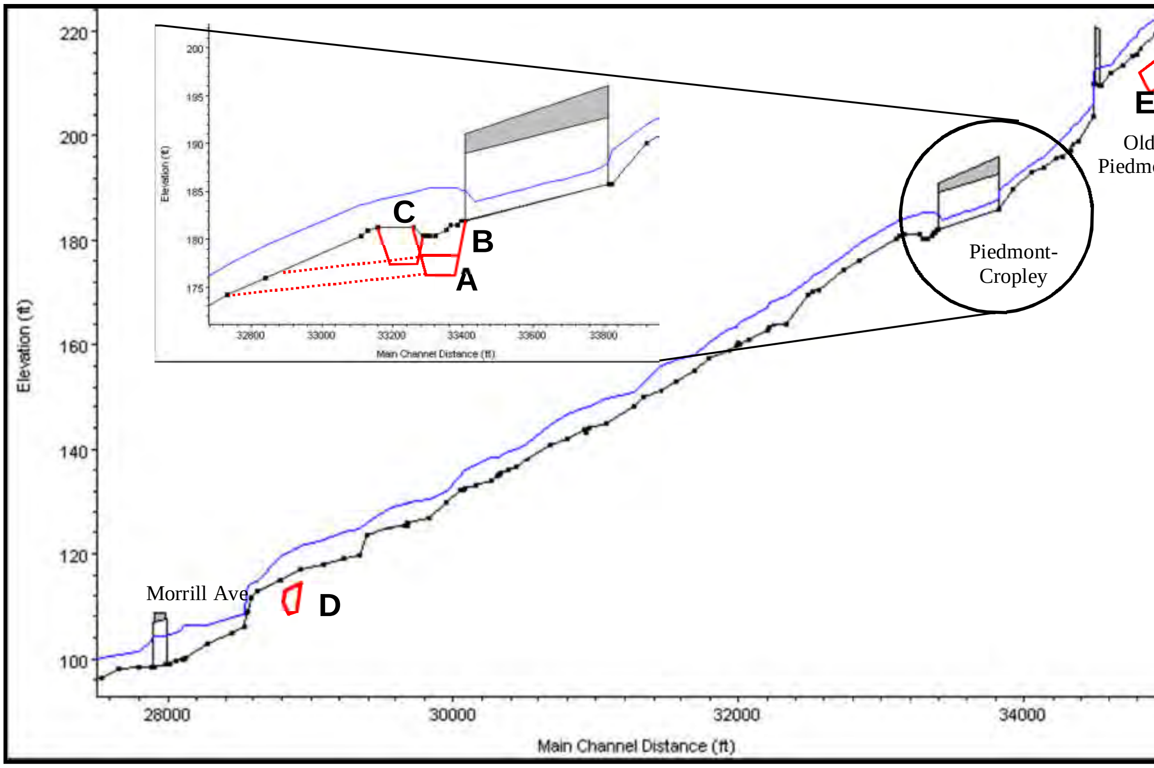

Figure 3-14 Profile View of Alternative Sediment Basin Configurations

CHAPTER 4: RECOMMENDATIONS FOR ADDITIONAL ANALYSES

To support the further development of the preferred alternative once selected, additional analyses and investigations related to the determination of sediment transport conditions within the project area should be performed. These analyses will assist in refining the design and providing a project that functions properly in relation to geomorphic and sediment transport conditions. The recommended investigations and analyses include the following:

- Perform inspections of the major tributaries entering the project to assess their sediment contribution and whether there are opportunities for sediment management on the tributaries. Past studies have focused on the main Berryessa Creek drainage since it is the largest sediment source; however, some opportunities may exist to improve sediment transport conditions within the project by addressing the supply of sediment from the tributaries.

- The HEC-6T model developed for the without-project condition should be applied to with-project condition. The results from the without-project condition showed that the model reasonably predicts the locations of sediment deposition and scour. The following are specific recommendations for the HEC-6T effort:

  - The model should be developed as an assessment and design tool for the preferred alternative rather than being applied in the alternative selection process. Application of the sediment transport and geomorphic assessment presented in this report should be adequate during the plan selection effort.

  - The current model uses only one sediment size distribution for the entire project area. This assumption should be reviewed and the possibility of utilizing several distributions as conditions change should be evaluated. This should be considered in terms of both the surface and subsurface distributions.

  - Based on the review of the NHC (2003) report, it did not appear the sediment removal was incorporated into the modeling effort. Consideration of running multiple events and incorporating sediment removal should be considered.

  - In applying the HEC-6T model some thinning of cross sections may be necessary from those used in the current HEC-RAS hydraulic model.

- Further refinement of the project design in terms of the channel sections should be undertaken to reduce the wide variations in velocities that occur within short distances. Many of these rapid variations may be due to the concentration of the initial design effort on determining the levee heights and bridge modifications to contain the design floods. The initial design modifications addressed the channel cross section size and levee heights primarily. In the next level of design, some adjustment of the channel gradient may be incorporated to provide a design with more consistent hydraulic conditions.

- Design modifications for the alternatives at several of the bridges downstream of I-680 result in increased flow areas that consequently cause existing deposition trends to be exacerbated. Specific problem areas identified are at Calaveras Boulevard, the UPRR trestle and Montague Expressway.

- Scour analyses need to be conducted to determine toedown depths for toe protection. General scour from the HEC-6T analysis should be added to bend and toe scour estimates. Because of the many modifications at bridges, the adequacy of the piers and abutments must also be evaluated in terms of scour, both local and general.

- Sizing of bank protection needs to be undertaken. Additionally, the ability of the upper bank protection and the vegetation on the floodplains to prevent erosion needs to be assessed based on shear stress and velocities.

- The n-values (roughness coefficients) assigned to the various channel components need to be adjusted if further refinements are made in terms of decisions on the types of vegetation that will be established in each area.

- Further analysis of potential changes in the configuration of the Piedmont sediment retention basin and other sediment retention facilities upstream of Old Piedmont Road need to be performed to quantify sediment removal.

- A more quantitative comparison should be made between these sediment modeling results and other modeling carried out by Jordan (2009) using SIAM and GSTARS-1D where possible, to reinforce confidence in model results.

CHAPTER 5: REFERENCES

Jordan, B., 2009. An Urban Geomorphic Assessment of the Berryessa and Upper Penitencia Creek Watersheds in San Jose, California. Colorado State University, Fort Collins, CO.

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CHAPTER 6: ADDENDUM 1

6.1 Summary and Excerpts from Colorado State University Doctoral Dissertation

A detailed study comparing Berryessa Creek with Penitencia Creek was conducted as part of a PhD dissertation by Brett Jordan at Colorado State University. Full citation information and a summary of parts of the dissertation most pertinent to this study prepared by Tetra Tech, Inc. are presented in the following paragraphs.

Jordan, B. (2009). *An Urban Geomorphic Assessment of the Berryessa and Upper Penitencia Creek Watersheds in San Jose, California*. Colorado State University, Fort Collins, CO.

6.1.1 Summary of Abstract

- A quantitative urban geomorphic assessment was conducted for the Berryessa Creek watershed to investigate the effects of urban hydrologic change, valley subsidence and river infrastructure elements on channel stability.
- 47 monumented cross sections over a 3000-meter reach of Berryessa Creek were surveyed in 2004. Cross sections were surveyed yearly after high flow season (winter) for 3 years to document changes in river processes and form.
- Detailed geomorphic field data were used to conduct hydrologic and sediment transport modeling and investigate the relative effects of hydrologic alteration, valley subsidence and river infrastructure on water yield, sediment yield and channel stability.
- Results of this analysis indicate system instability in the urbanized valley portion of Berryessa Creek is caused primarily by drainage area capture by the urban storm sewer network and engineered river infrastructure elements.
- Hydrologic and sediment modeling indicates that these drainage system modifications have caused a water yield increase of 48 % and sediment yield increase of 9 % to 61 % based on historic conditions.
- Changes in the Berryessa Creek hydrological regime have transformed previously depositional reaches into incised reaches. Results of modeling indicate the maximum incision due to valley subsidence would be 0.27 m.
- Effects of base level lowering will be at a maximum approximately 500m upstream of the zone of maximum subsidence, which is minor increase in sediment yield of 0.3 % to 11 %. River infrastructure (an online sedimentation basin and 1.85 m grade control structure) has reduced the downstream sediment yield by 15 %.
- Subsidence effects from groundwater extraction are obscured by current channel instability caused by urban development which dominate system changes.

6.1.2 Summary of Introduction

- Methods of analysis: 1. time series aerial photos, topographic data, long profile analysis. 2. Field data collection. 3. Numerical hydrology and sediment transport modeling.

- The Berryessa watershed is an alluvial fan that has been anthropogenically manipulated along the valley floor to facilitate agriculture and urban development.
- Berryessa has been subject to channel realignment, engineering infrastructure, floodplain encroachment, drainage area expansion via storm sewers and has suffered severe erosion and sedimentation problems (e.g. in Summer 2004 approximately 7,100 m^3 sediment was dredged from two reaches of Berryessa; in comparison there was very little removal of sediment from fish ladder structures on the less modified Penitencia Creek).
- This dissertation contains a large literature review about effects of urbanization on watershed hydrology, sediment transport and ecology.
- Land subsidence of up to 3.5m was observed in parts of the Santa Clara Valley between 1934 and 67 due to groundwater pumping.

6.1.3 Summary of Methodology

- Page 29 contains useful table of all data collected.
- The study examined a time series of long profiles. Berryessa Creek has undergone 1.5m or more incision or mechanical sediment removal in reach where the steep upland transitions in valley flat, this reach would be expected to be depositional. The reason for this is channelization and floodplain encroachment.
- Page 36 presents the change in bed level over time. More scour than deposition is evident on Berryessa Creek.
- Historical aerial photography analysis showed in 1899 there was no defined channel on Berryessa Creek below mountain range, just the alluvial fan with multiple small paths. By 1939 the single thread channel had been formed by channelization to permit agriculture on the fan, development and flood control. Lengthening of the channel decreased the slope significantly. In 1899 it was 0.02, 1930s it was 0.01, 1950s it was 0.005. The natural stream response of reducing the gradient was to aggrade.
- Subsidence by reach on Berryessa: Reach 1: 1125-2000: 0.11m, Reach 2: 710-1125: 0.14m, Reach 3: 250-710: 0.23m. Normal base-level lowering causes increase in sinuosity. Conversely an increase in urbanization normally results in decrease in sinuosity due to lateral restraints and channelization.
- Reach 1: most upstream. Between 1939 and present a decrease in sinuosity due to channelization 1960-80 is observed. Reaches 2 and 3: no channelization has taken place, trend of increased sinuosity, likely due to increased discharge and reduced sediment load.
- Similar trends were observed in the meander belt width.
- Urbanization mainly occurred in the valley areas between 1960s and 1980s; little urbanization has taken place in the upper watershed.
- A drainage area expansion took place on Berryessa due to addition of two historic alluvial fan streams. In 1899 the drainage area was 13.0 sq km, in 2002 it was 15.5 sq km.
- The watershed is located on active Hayward fault. Large landslide activity delivers large sediment load to channel.
- Previously change in valley grade from steep uplands to flatter valley means sediment is deposited at interface. Berryessa sediment basin was constructed in 1962 has

reduced sediment deposition and can easily be excavated but sediment continuity downstream has been disrupted.

- Sediment has been dredged every 2 years between 1984 and 2004. The basin is effective at capturing large particles (>16 mm) transported as bedload. This has caused channel incision downstream.
- Summary: Upper Berryessa watershed is not urbanized, the lower watershed has become 85 % urbanized over last 100 years. Changes in hydrology magnify peaks and duration of flows capable of producing bedload transport in Berryessa Creek. A trend for downgrading and incision has been observed. (1.5m of incision between 1967 and 2004 downstream of the sedimentation basin). Berryessa has only subsided 0.23m (Penitencia 1.1m).
- Cross sections were resurveyed and the average bed change was calculated. Over 65% of Berryessa cross sections are degrading.
- Manning's n for Berryessa was considered to range between 0.037 and 0.064, with a mean of 0.047.
- Pebble counts conducted at each cross section. Page 89 contains a bed material size plot over the long profile.
- Bulk sampling was carried out. Berryessa shows fining (as would be expected) moving downstream. There is a sharp drop in size after the sediment basin as coarse particles are trapped in the sediment basin.
- Bank condition reconnaissance was carried out and the following sediment properties were recorded: depth of layer, sphericity (round, angular), texture, color, clast matrix supported structure, grain size, sorting.
- Bank height and angles were measured visually for stable and unstable bends. Bank height to depth ratio has been proposed as a measure of stability.
- Erosion pins (referred to as "bank rods") were installed for the winter 2004 season and monitored until 2006. Bank retreat ranged from 0 to 0.36m/yr.
- Bank material varies considerably between stratigraphic units.
- 15 min stage and discharge data was collected in 2005 and 2006. Bedload and suspended load were measured to develop a rating curve. Bedload sizes were measured at two locations on Berryessa.
- Rating curves for bedload and suspended were developed, although plots exhibit a considerable amount of scatter even with log-log axes. Comparing Berryessa to Penitencia, Berryessa has much large supply of sediment than Penitencia. Upland reaches of Berryessa have a considerable amount of landslide activity and colluvial sediment sources.

6.1.4 Hydrological Modeling

- Processes that have lead to flow regime changes on Berryessa Creek include increase in watershed impervious area and increased connectivity/changes in catchment area.
- A calibrated hydrological model was created in HEC-HMS. Three different simulations carried out.
- Upper watershed is characterized by steep slopes, clay/gravely loam soils with low infiltration rates. The valley has low relief, sandy soils and higher infiltration rates.

- Urbanization in the Berryessa watershed has caused a net increase of 14 % in urbanized land use for whole watershed. Diversions have created a 20 % increase in effective catchment area, causing higher peak flows and volumes.
- Hydrographs currently have higher peak discharges and more flashy time to concentration due to efficiency of the storm drains than historical conditions, resulting in multiple peaks for an event that would previously have a single peak.

6.1.5 Sediment Transport Modeling

- Two sediment transport models were used to evaluate urbanization and valley subsidence effects on channel stability: SIAM (snapshot in time) and GSTARS-1D (continuous simulation used to predict long term channel changes).
- Six versions of each model were produced for Berryessa Creek: two different geometries – historic (1939), current (2004) with urban infrastructure, current (2004) without urban infrastructure.
- As part of the dissertation efforts, a HEC-RAS model was developed by Colorado State University (CSU) independently from the Corps of Engineers model. The CSU HEC-RAS model was used to create the SIAM model. Ten SIAM reaches were used.
- A sediment transport function sensitivity analysis was carried out. Ten equations were tested. The synthesized results were compared with measured suspended load and bedload data, and observed morphology changes. Yang (1973) and Yang (1984) appeared to be most accurate and were selected for model use.
- 30-year simulations carried out with GSTARS-1D. The models do not include subsidence.
- Model results were compared to field observations. SIAM produced results closer to observed results than GSTARS-1D. Both models provide reasonably close predictions. SIAM showed a good agreement with amount of sediment deposited in the Berryessa basin on annual basis (compared against the dredging records).
- Models indicate that the watershed changes on Berryessa would induce significant channel change, especially in downstream reaches: change from deposition to incision, increase in sediment yield.
- Models indicate that instability problems may be introduced to the upstream reaches by removing the grade control structure on Berryessa Creek: degradation upstream, aggradation downstream.

6.1.6 Appendices

- Bankfull dimensions by cross section, superimposed surveyed cross sections from 2004/2005/2006 and bed material size data are presented.