Final Report

San Francisquito Creek Development and Calibration/Verification of Hydraulic Model

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May 26, 2009

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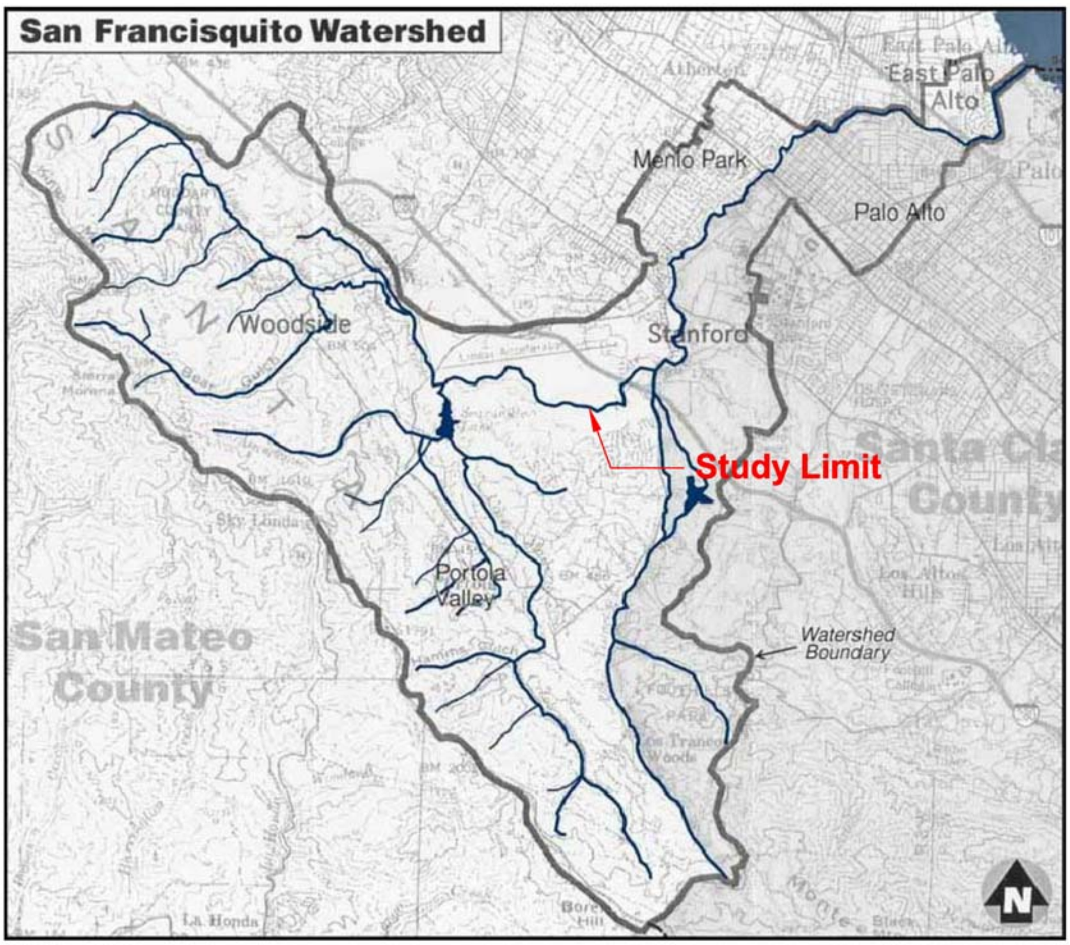
1 INTRODUCTION

The San Francisquito Creek watershed encompasses an area of approximately 45 square miles, extending from the ridge of the Santa Cruz Mountains to the San Francisco Bay in California. San Francisquito Creek begins at the confluence of Corte Madera Creek and Bear Creek at Jasper Ridge Preserve of Stanford University and flows into San Francisco Bay approximately 2.5 miles south of the Dumbarton Bridge. The location of the San Francisquito Creek and the watershed map are sown in Figure 1-1. San Francisquito Creek has an inadequate carrying capacity due to development, vegetation, sedimentation, land subsidence, levee settlement and erosion. Flooding on the creek affects the cities of Menlo Park and East Palo Alto in San Mateo County, and Palo Alto in Santa Clara County.



Source: San Francisquito Watershed Council Website, modified by NCI

Figure 1-1. San Francisquito Creek Vicinity Map

For the purpose of evaluating the flow capacities of the existing channel and the major bridges on the creek, a one-dimensional HEC-RAS model was developed for San Francisquito Creek. The modeled reach is from the mouth of the creek to approximately one mile upstream of Highway 280, with a total length of approximately 55,000 feet. This hydraulic model can further be used to assess future flood conditions, delineate floodplains, and assist in the development of alternatives for improved capacity and regional flood reduction.

The geometric data of the model was developed based on the channel topographic survey (BE, 2008) and the LiDAR for the project area, both provided by the U.S. Army Corps of Engineers (USACE). After creating a Digital Terrain Model (DTM) by merging the field channel survey results with the LiDAR data, the HEC-GeoRAS program was used to derive the channel geometric data for import into HEC-RAS, where the geometric data was refined and completed, including the incorporation of the bridge information.

After calibrating and verifying the HEC-RAS model using three historic flood events, the model was used to simulate other four historic flood events that caused flooding of the creek. The existing flow capacities of the creek and bridges were evaluated based on the model simulations for a series of flow discharges. A sensitivity analysis was also conducted to test how the Manning's roughness coefficient impacts the predicted flow capacities.

2 MODELED SCENARIOS

A series of scenarios have been simulated with the HEC-RAS model. There are 5 plans in total that were formulated in the model, as summarized in Table 2-1. Each plan consists of multiple scenarios. Each scenario corresponds to the combination of one channel geometric condition and one flow condition.

Plan 1 is for the model calibration and verification using three historic flood events with the high water marks being measured: the February 13, 2000 flood event (SCVWD, 2009), the December 16, 2002 flood event, and the January 1, 2006 flood event (JPA, 2009). Plan 2 includes other four historic flood events that have caused significant creek flooding. Different from the three flood events used in the model calibration/verification, only the flooding locations instead of the high water marks were documented for these four flood events in Plan 2.

Plan 3 consists of 50 scenarios, with 50 synthetic flow discharges, ranging from 3,500 cubic feet per second (cfs) to 8,800 cfs, being modeled with the existing channel geometry. The 100-year flow rate of San Francisquito Creek was estimated to be 8,800 cfs at the USGS 11164500 Station (Wang et al., 2007). The purpose of Plan 3 is to determine the flow capacities of the existing channel and bridges. The purpose of Plans 4 and 5 is for the sensitivity analysis. The same 50 flow discharges were modeled in these two plans. Compared to Plan 3, the Manning’s roughness values were reduced by 0.005 in Plan 4 and increased by 0.01 in Plan 5.

Table 2-1. HEC-RAS Modeled Scenarios

Plan	Scenarios
1	Three historical flood events: 2/13/2000, 12/16/2002, 1/1/2006.
2	Four historical flood events: Dec 1955, Apr 1958, Jan 1982, and Feb 1998.
3	50 synthetic flow discharges between 3,500 cfs to 8,800 cfs with downstream MHHW.
4	Same flow conditions as Plan 3, but reducing Manning's roughness values by 0.005.
5	Same flow conditions as Plan 3, but increasing Manning's roughness values by 0.01.

3 HEC-RAS MODEL DEVELOPMENT

The U.S. Army Corps of Engineers' River Analysis System (HEC-RAS) was developed by the Hydrologic Engineering Center. This software was designed to perform one-dimensional steady flow, unsteady flow calculations, sediment transport/mobile bed computations and water temperature modeling (USACE 2008). The steady flow component of HEC-RAS was used in the present hydraulic modeling study.

3.1 Description of HEC-RAS and HEC-GeoRAS Models

The steady flow component of HEC-RAS is capable of modeling subcritical, supercritical, and mixed flow regime water surface profiles. The basic computational procedure is based on the solution of the one-dimensional energy equation. Energy losses are evaluated by friction (Manning's equation) and contraction/expansion (coefficient multiplied by the change in velocity head). The momentum equation is utilized in situations where the water surface profile is rapidly varied. These situations include mixed flow regime calculations, hydraulics of bridges, and evaluating profiles at river confluences. The effects of various obstructions such as bridges, culverts, weirs and other hydraulic structures can be included in the model computations.

Data input requirements for the HEC-RAS model (flow component) include (1) the geometric data for the river system and (2) the flow data and boundary conditions. The geometric data includes the river system connectivity (schematic), cross-section data (geometry, Manning's roughness, contraction/expansion losses, ineffective flow areas, etc.), and hydraulic structure data (bridges, culverts, dams, weirs, etc.). The flow data includes the flow discharges at the upstream end of a reach and at the flow change locations. The boundary conditions are necessary to establish the starting water surface elevation at the ends of the river system. In a subcritical flow regime, boundary conditions are only required at the downstream ends of the river system.

HEC-GeoRAS is a set of the ArcGIS tools specifically designed to process geospatial data for users with HEC-RAS. HEC-GeoRAS creates a file of geometric data for import into HEC-RAS and enables viewing of exported results from HEC-RAS. The geometric file is created from data extracted from data sets (ArcGIS layers) and from a Digital Terrain Model (DTM). Hec-GeoRAS requires a DTM represented by a triangulated irregular network (TIN) or a GRID.

3.2 Channel Geometry

The channel geometry used in the hydraulic model was developed based on a channel topographic survey (BE, 2008) and the LiDAR for the project area. Both the field survey results and the LiDAR were provided to us by the U.S. Army Corps of Engineers (USACE). After creating a DTM by merging the field survey results with and the LiDAR data, the HEC-GeoRAS program was used to derive the channel geometric data for import into HEC-RAS, where the geometric data was refined and completed, including the incorporation of the bridge information.

3.2.1 Development of DTM

A TIN DTM was developed in this study to represent the channel topography for the project area based on the field survey results and LiDAR data. The channel topographic survey was conducted by Bestor Engineers, Inc. (BE) and Sea Surveyor, Inc (SSI) from October 2007 through August 2008. This field survey covers the entire reach of the San Francisquito Creek that was modeled in this study. However, this survey only measured the main portion of the cross sections along the creek, not the full extent of the cross sections for some areas. As an alternative, the LiDAR was used to provide the topographic information for these areas.

The field survey data was provided in AutoCAD format, in which the (topographic) surface of the survey data points was already created. This surface was then loaded in ArcGIS as polylines to create the TIN for the channel based on the field survey.

The regional LiDAR data were provided in LAS format with an estimate of more than 100 million points. A subset was developed based on the regional LiDAR dataset. The subset covers an area along the San Francisquito Creek, and extends laterally 500 feet from the left and from the right boundaries of the field channel survey coverage. The area outside of the subset coverage, which is more than 1,000 feet in width, is beyond the interest of area in this model study. The LiDAR subset and the channel TIN were then merged, by replacing the LiDAR data in the areas covered by the channel TIN. The final DTM covers an area along the modeled reach of the creek. The width of the DTM coverage is more than 1,000 feet wide with the creek lying along the centerline. In order to facilitate the processing time, the final DTM of the project area was represented by two TINs, one for the area downstream of El Camino Real Bridge, and the other for the area upstream.

The horizontal coordinate system for the HEC-RAS model and DTM is the California State Plane NAD83, Zone 3, in US Survey Feet. The vertical datum is NAVD88, feet. All Model boundary conditions and output will be referenced to this datum.

3.2.2 Development of Channel Geometry with HEC-GeoRAS

The channel geometry data was created from the TIN DTM using HEC-GeoRAS. The RAS layers created include the Stream Centerline, Bank Lines, Flow Path Centerlines, XS Cut Lines and Bridges/Culverts. The Stream Centerline was digitized along the centerline used in the Bestor and Sea Surveyor (2008) field survey, with minor modifications in a few places based on the channel topography. Therefore, the river stationing of the HEC-RAS model is very close to the field survey. There are 381 cross sections in total to represent the modeled reach of approximately 55,000 feet long.

The geometry data in the HEC-GeoRAS's output mainly includes the stream centerline (river system schematic), geometry of cross sections, and locations and preliminary deck information of the bridges.

3.2.3 Refining of Cross-Sectional Geometry

After importing the geometry data created with HEC-GeoRAS into HEC-RAS, the data were further completed and refined. These include (1) adding Manning's roughness values, (2) adding levee data, (3) filtering/adjusting cross-section points and adjusting bank stations if necessary, and (4) completing the bridge data.

The cross-sectional geometry derived with HEC-GeoRAS was compared to the field topographic survey data (drawings) with good agreement found. However, the cross sections derived based on the DTM are generally wider than the cross sections that could be derived from field survey alone.

3.3 Bridges

There are 13 bridges and 10 pedestrian bridges identified in the Bestor (2008) field survey. All of the 23 bridges were incorporated into the HEC-RAS model. After importing the geometric data created with HEC-GeoRAS into the HEC-RAS model, the preliminary bridge data was updated and/or completed based on the bridge dimensions as shown in the Bestor (2008) field survey. The updated or new bridge data includes the bridge deck information (width of the bridge, high chord and low chord profiles, etc) and bridge pier data. It is noted that these 23 bridges show a significant difference in types, deck elevations and opening dimensions. As a result, the flow capacities of these bridges also vary significantly, as found in this hydraulic model study.

3.4 Flow Data and Boundary Conditions

The flow data and boundary conditions include (1) the flow discharges at the upstream end of the creek and at the flow change locations, and (2) the water surface elevations at the mouth of the creek.

San Francisquito Creek has two major tributaries: Bear Creek and Los Trancos Creek. The confluence of Bear Creek is beyond the modeled reach. The confluence of Los Trancos Creek is approximately 47,000 feet upstream of the creek mouth, or 8,000 feet downstream of the upstream limit of the study area. Based on the hydrology study (Wang et al., 2007), the flow discharge of Los Trancos Creek is approximately 11 percent to 14 percent of that for San Francisquito Creek downstream of the confluence. In another words, the flow discharge in the upper San Francisquito Creek (upstream of the confluence of Los Trancos Creek) is approximately 86 percent to 89 percent of the reach downstream. In the HEC-RAS model, the flow rate upstream of the confluence was assigned to be 87 percent of the downstream reach. It is noted that different assignments of the flow ratio would only impact the model results for the reach upstream of the confluence, which is less than 15% of the study reach and which has no flooding potential for the 100-year flood event.

The flow discharge of San Francisquito Creek generally increases from upstream to downstream as a result of the increasing drainage areas, as listed in Table 3-1. The estimated 100-year flow increases from 8,800 cfs at the USGS 11164500 Station to 9,400 cfs at Palo Alto

Airport of Santa Clara County, or approximately 7 percent increase. While the confluence of Los Trancos Creek was included in the model as the flow change location, the flow increase that is solely caused by increasing drainage area without a tributary inflow was, however, not considered in the model. It is noted that the flow discharges presented in this report are referred to the USGS 11164500 Gage Station.

Table 3-1. Peak Flow Rates Along San Francisquito Creek

Locations	Drainage Area (mi^2)	10-Year (cfs)	100-Year (cfs)
Upstream of Los Trancos Creek	29.61	3,900	7,600
Downstream of Los Trancos Creek	37.26	4,500	8,800
USGS 11164500 Station	**37.62**	**4,500**	**8,800**
At El Camino Real	41.20	4,700	9,200
At US Hwy 101	44.55	4,800	9,300
At Palo Alto Airport of Santa Clara County	46.17	5,000	9,400

Sourece: Santa Clara Valley Water District, San Francisquito Creek Hydrology Report, Prepared by Wang et al. Revised December 2007.

While a series of synthetic flow discharge events were modeled in order to determine the flow capacity of the existing creek and bridges, several historical flood events were used to calibrate and validate the HEC-RAS model. The model was calibrated and verified using the three flood events that occurred on February 13, 2000, December 16, 2002, and January 1, 2006, respectively. The flow discharges and the high water marks measured along the creek were provided by Santa Clara Valley Water District for the February 13, 2000 flood event, and by the San Francisquito Creek Joint Powers Authority (JPA) for the other two flood events. The flow discharges for the other four historical flood events, which have caused significant flooding, are presented in the Reconnaissance Investigation Report of San Francisquito Creek that was prepared by the CRMP Flood and Erosion Control Task Force (1998). The flow discharges for the modeled scenarios are summarized in Table 3-2.

The downstream water surface elevation at the mouth of the San Francisquito Creek was represented by the tidal stage in South San Francisco Bay. Among the tidal level stations administrated by the National Oceanic and Atmospheric Administration (NOAA), the Redwood

City Station (Station ID: 9414523) is the station that is closest to this creek mouth and that has a long term record of measured tidal data. Therefore, the tidal stage information at this station was used to represent the water level at the mouth of San Francisquito Creek. For the historical flood events, the water levels at the creek mouth were assigned to be the highest tidal elevations measured at the NOAA Redwood City Station during these flood events, as summarized in Table 3-2. The Mean Higher High Water (MHHW) elevation at the Redwood City Station, +7.1 feet, NAVD88, was used as the water level at the creek mouth for the flow capacity assessment.

Table 3-2. Flow Data and Downstream Tidal Stages for Modeled Scenarios

Plan	Flood Events	Flow Discharge [a] (cfs)	Tidal Stage [e] (feet, NAVD88)
1	February 13, 2000	4,010 [b]	+7.3 [f]
	December 16, 2002	3,730 [c]	+8.3 [f]
	January 1, 2006	4,840 [c]	+8.9 [f]
2	December 1955	5,560 [d]	+7.9 [f]
	April 1958	4,460 [d]	+8.5 [f]
	January 1982	5,220 [d]	+8.5 [f]
	February 1998	7,100 [d]	+9.3 [f]
3	50 Synthetic Flood Events	3,500 ~ 8,800	+7.1 [g]
4	50 Synthetic Flood Events	3,500 ~ 8,800	+7.1 [g]
5	50 Synthetic Flood Events	3,500 ~ 8,800	+7.1 [g]

Note: [a] Flow Discharge at USGS 11164500 Gage Station.

[b] Provided by Santa Clara County Water District During the Kickoff Meeting.

[c] Provided by San Francisquito Creek Joint Powers Authority (JPA).

[d] Presented in CRMP Flood and Erosion Control Task Force (1998) Report.

[e] Tidal Elevation at NOAA Redwood City Station (NOAA 9414523).

[f] Highest Tidal Elevation Measured at Redwood City Station during the Flood Events.

[g] Mean Higher High Water (MHHW) at Redwood City Station.

It is noted that a conversion of 2.75 feet was used to convert NGVD29 to NAVD88 (0 feet NGVD29 = +2.75 Feet NAVD88). The same conversion value is being used in Santa Clara County and for the South San Francisco Bay Shoreline Study.

4 HEC-RAS MODEL CALIBRATION AND VERIFICATION

The HEC-RAS model was calibrated by adjusting the Manning's roughness values to obtain a reasonable agreement between the predicted water surface profile and the measured gage data during the flood events. Three flood events, with the flow discharges and the high water marks along the creek being measured, were used to calibrate and verify the model. These three flood events occurred on February 13, 2000, December 16, 2002, and January 1, 2006, respectively. The flow data were provided by Santa Clara Valley Water District (SCVWD, 2009) for the February 13, 2000 flood event, and by the San Francisquito Creek Joint Powers Authority (JPA, 2009) for the other two flood events. These three flood events were used for model calibration because (1) the estimated flow rates are believed to have better quality, and (2) the high water mark elevations were recorded at multiple locations (instead of one location) along the creek so that the model can be calibrated in more detail.

The comparisons of the water surface elevations between the model results and the measured data are summarized in Table 4-1 for these three flood events. The comparisons are also shown in Figure 4-1 through Figure 4-4 for the four locations where the high water marks were recorded. The predicted water levels show good agreement with the measured data for all the three flood events at three of the four locations: Hwy 101 Bridge, Waverley Bike Bridge, and the USGS Gage Station. At the Pope/Chaucer Bridge, the discrepancy between the predicted water level and the measured high water mark is 0.5 feet for the December 16, 2002 flood event and 0.6 feet for the January 1, 2006 flood event, but is as much as 2.1 feet for the February 12, 2000 flood event. It is noted that the measured flow data at this location shows inconsistence between the flood events. While the flow rate for the December 16, 2002 flood event was less than the February 13, 2000 flood event, the measured high water mark for the December 16, 2002 flood event is 1.6 feet higher than the February 13, 2000 flood event. If there was nothing abnormal occurred near this location during these two flood events, the accuracy of the measured high water mark data is questionable, which will contribute to the discrepancy between the model results and data found at this location.

The calibrated Manning's roughness values show a consistent variation trend along the creek, as summarized in Table 4-2. For the main channel, it ranges between 0.03 for the hydraulically smoother reach downstream of Station 87+30 (approximately 850 feet upstream of the Hwy 101

Bridge centerline), 0.04 for the reach between Station 87+30 and the Newell Road Bridge, and 0.043 for the hydraulically rougher reach that is upstream of the Newell Road Bridge. The floodplain roughness coefficients are generally larger than the main channel by 0.02 for each reach. It is noted that the floodplain is very limited for most of the creek, and therefore the floodplain roughness values do not have significant impact on the predicted water surface profiles.

Table 4-1. Comparison of Water Level between Data and Model Results

Location	Representative Sta. in HEC-RAS Model	Water Levels (ft, NAVD88)					
		2/13/2000 (4,010 cfs)		12/16/2002 (3,730 cfs)		1/1/2006 (4,840 cfs)	
		Data	Model	Data	Model	Data	Model
Upstream of Hwy 101	Sta. 80+27	+16.9	+16.9	+16.2	+16.5	+18.2	+18.1
Pope/Chaucer Bridge	Sta. 178+37	+38.1	+40.2	+39.7	+39.2	+43.4	+42.8
Waverley Bike Bridge	Sta. 249+00	+55.9	+55.7	+55.3	+55.1	+57.4	+57.5
USGS Gage Station	Sta. 405+61	+121.8	+122.1	-	+121.7	-	123.2

Table 4-2. Calibrated Manning's Roughness Values

Reaches	Manning's Roughness Values	
	Main Channel	Floodplain
Downstream of Sta 87+30 (850' u/s of Hwy 101 Bridge)	0.03	0.05
Sta 87+30 to Newell Road Bridge (Sta 112+23)	0.04	0.06
Upstream of Newell Road Bridge (Sta 112+23)	0.043	0.063

The lower portion of San Francisquito Creek approximately downstream of Newell Road Bridge can be characterized as a natural (tidal) stream: some vegetation with the creek bottom consisting of bay mud, sand and cobbles. The Manning's roughness value for this type of creek is recommended to be from 0.025 to 0.04 in the HEC-RAS User's Manual (USACE, 2008). The reach upstream of Newell Road Bridge shows characteristics of mountain creeks: gravels, cobbles, a few boulders on the channel bottom, and steep banks with trees and brush on submerged bank. The Manning's roughness value for this type of creek is recommended to vary between 0.03 and 0.05. It is noted that the roughness coefficients calibrated in this study are consistent with the values recommended by the HEC-RAS User's Manual (USACE, 2008).

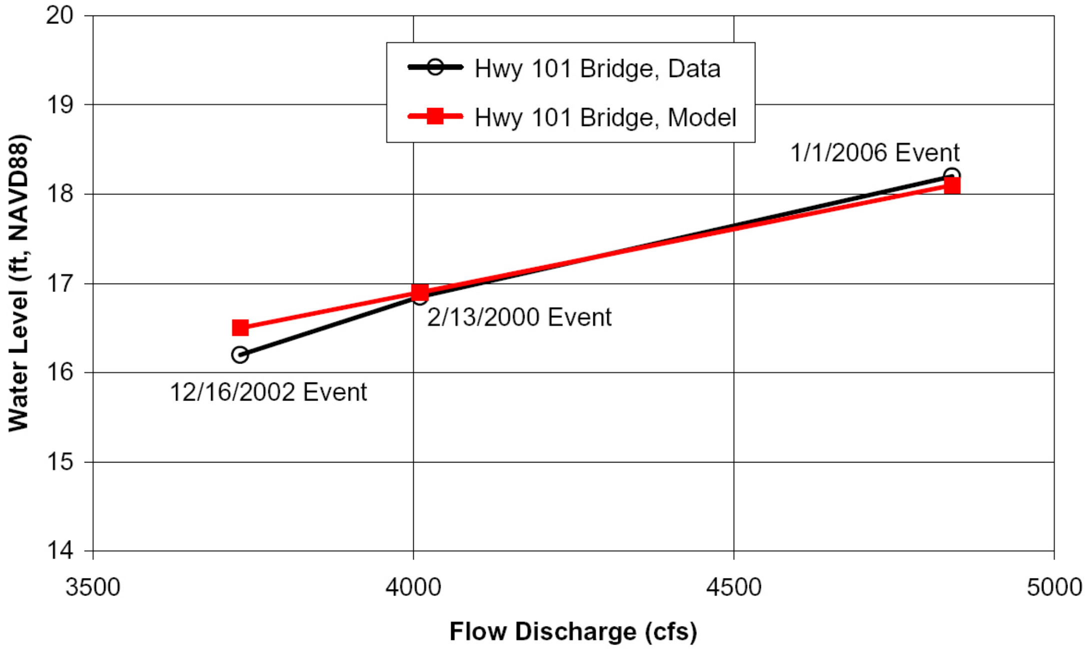


Figure 4-1. Predicted Water Levels versus Data at the Hwy 101 Bridge

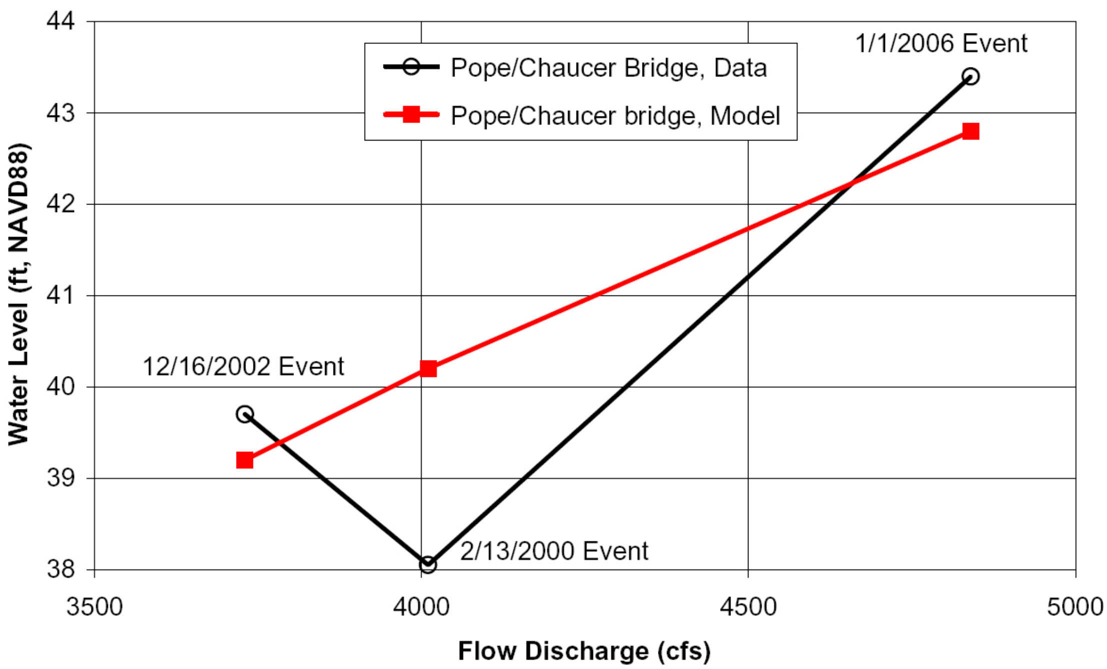


Figure 4-2. Predicted Water Levels versus Data at Pope/Chaucer Bridge

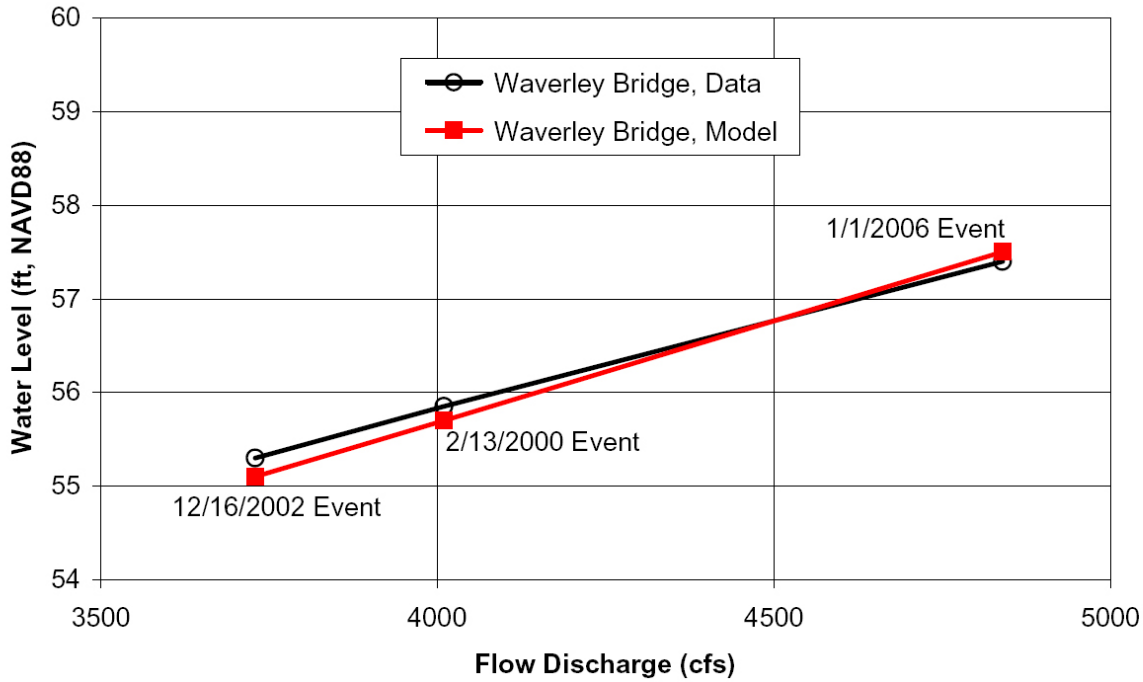


Figure 4-3. Predicted Water Levels versus Data at Waverley Bike Bridge

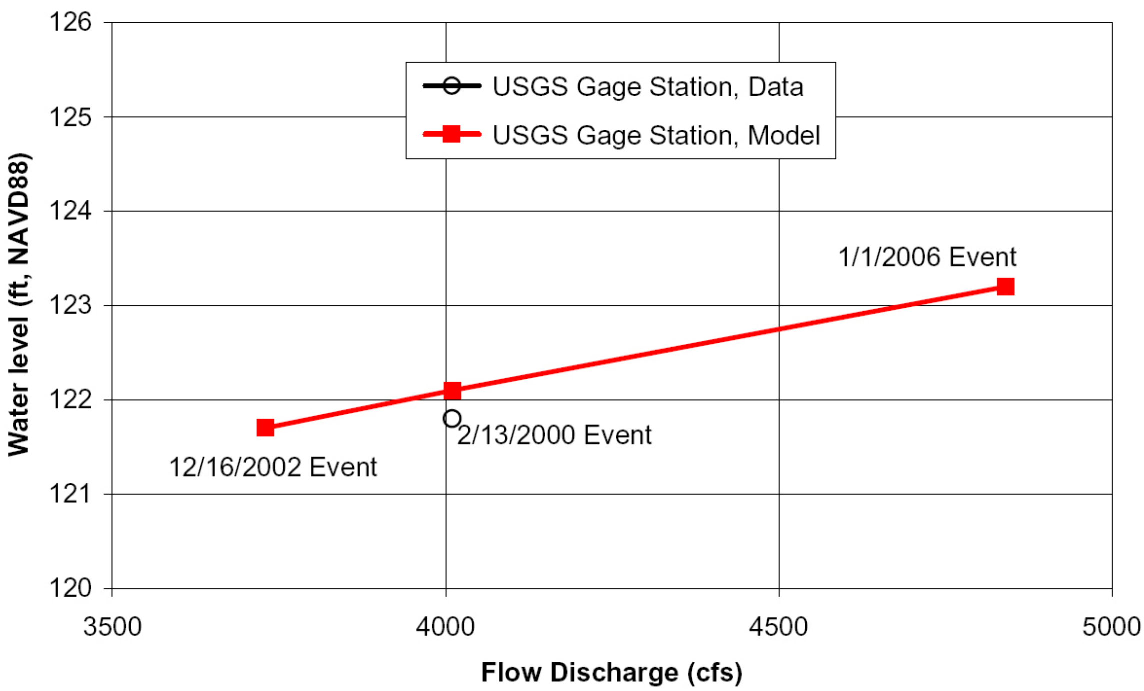


Figure 4-4. Predicted Water Levels versus Data at USGS Gage Station

5 VERIFICATION OF OTHER HISTORICAL FLOOD EVENTS

According to the CRMP (1998) reconnaissance report, the recent flood events that occurred in December 1955, April 1958, January 1982, and February 1998, respectively, caused significant flooding at several locations along San Francisquito Creek. Although no water level data was presented in the CRMP (1998) report for these flood events, the flooding locations were documented. The HEC-RAS model has been calibrated and verified using the three historic flood events with measured water level data, as discussed in Section 4. However, these additional four flood events were also modeled in this analysis for the purpose of additional verification of the model. It noted that the comparison between the model results and field observation for these four flood events is limited only to the flooding locations rather than the water levels due to the data unavailability. This verification is qualitative rather than quantitative.

By comparing the predicted water surface profiles with the creek bank elevations and with the deck elevations of the bridges, the modeled flooding locations were estimated for these four historical flood events, as summarized in Table 5-1. The predicted water surface profiles for the reach downstream of El Camino Real Bridge are shown in Figure 5-1 through Figure 5-4 for these flood events, respectively. The elevations of the left bank and right bank (represented by left levee and right levee) are also shown in these figures.

The predicted flooding locations for these four historical flood events are generally in agreement with the field observations as presented in the CRMP (1998) reconnaissance report. It is noted that the model simulations are based on the existing channel geometry which can be different from the channel conditions when these historical flood events occurred. The minor discrepancy between the predicted flooding locations and the field observation is most likely caused by the difference in the existing geometry data being used in the model simulations and the historic channel/bridge conditions.

Table 5-1. Predicted Flooding Locations for Historical Flood Events

Flood Events	Flooding Locations
Dec 1955 (5,560 cfs)	▪ At bridges: Hwy 101, and Pope/Chaucer St ▪ Creek downstream of Hwy 101 Bridge ▪ Creek between Hwy 101 Bridge and Newell Rd Bridge ▪ Creek immediately upstream of Pope/Chaucer St Bridge
Apr 1958 (4,460 cfs)	▪ At bridge: Hwy 101 ▪ Creek downstream of Hwy 101 Bridge ▪ Creek between Hwy 101 Bridge and Newell Rd Bridge
Jan 1982 (5,220 cfs)	▪ At bridge: Hwy 101, and Pope/Chaucer St ▪ Creek downstream of Hwy 101 Bridge ▪ Creek between Hwy 101 Bridge and Newell Rd Bridge
Feb 1998 (7,100 cfs)	▪ At bridges: Hwy 101, Newell Rd, University Ave, Pope/Chaucer St, and Middlefield Rd ▪ Creek downstream of Hwy 101 Bridge ▪ Creek between Hwy 101 Bridge and Newell Rd Bridge ▪ Creek between Newell Rd Bridge and University Ave Bridge ▪ Creek between University Ave Bridge and Pope/Chaucer St Bridge ▪ Creek between Pope/Chaucer St Bridge and Middlefield Rd Bridge

6 EXISTING FLOW CAPACITIES OF THE CREEK AND BRIDGES

A series of 50 synthetic flood events were modeled in order to determine the flow capacity of the creek and bridges based on the existing channel geometry and bridge configurations. These 50 synthetic flood events represent 50 flow discharge scenarios combined with the MHHW at the creek mouth. The 50 flow discharges ranges from 3,500 cfs to 8,800 cfs. The 100-year flow rate of San Francisquito Creek was estimated to be 8,800 cfs at the USGS 11164500 Station (Wang et al., 2007).

It is noted that the flow discharge of San Francisquito Creek generally increases from upstream to downstream as a result of the increasing drainage areas, as discussed in Section 3.4. The confluence of Los Trancos Creek was included in the model as the flow change location. However, the flow increases solely caused by the increase in drainage area without tributary flow input was not considered in the model. For the purpose of simplicity and consistence, the flow discharges of San Francisquito Creek are referred to the USGS 11164500 Gage Station location throughout the analysis, including the model calibration/verification and the creek capacity assessment.

The flow capacity of the creek at a given location is defined as the maximum flow rate before the water overtops the local bank (local water surface elevation higher than the local bank elevation). The flow capacity of a bridge is defined as the maximum flow rate before the water overtops the bridge deck (local water surface elevation higher than the lowest point of the high chord of the bridge deck). By comparing the predicted water surface profile for the 50 synthetic flood events to the creek bank elevations or the (high chord) elevations of the bridge decks, the flow capacity was determined for the creek and the bridges. The estimated flow capacities are summarized in Table 6-1 for different locations along the creek, and in Table 6-2 for the major bridges. Figure 6-1 through Figure 6-11 show the predicted water surface profiles for the flood events corresponding to the flow capacities for different locations along the creek and for the bridges.

Table 6-1. Flow Capacities for San Francisquito Creek

Locations	Flow Capacity	Flooding Potential Rank
Adjacent to US 101 Bridge	4,400 cfs	1
Adjacent to Newell Road Bridge	6,200 cfs	4
Adjacent to University Avenue Bridge	5,800 cfs	3
Adjacent to Pope/Chaucer St Bridge	5,200 cfs	2
Adjacent to Middlefield Road Bridge	6,800 cfs	5
Adjacent to Piers Lane Bridge	8,200 cfs	6
Upstream of El Camino Real Bridge except for the portion adjacent to Piers Lane Bridge	Exceeding 8,800 cfs	7

Note: "Adjacent" means the reach within hundreds feet upstream and downstream of the bridge.

Table 6-2. Flow Capacities for Major Bridges

Bridges	Distance from Mouth (ft)[1]	Flow Capacity	Flooding Potential Rank
US 101 Bridge	7,900	4,700 cfs	1
Newell Road Bridge	11,200	6,500 cfs	3
University Avenue Bridge	13,400	6,800 cfs	5
Pope/Chaucer St Bridge	17,800	4,900 cfs	2
Middlefield Road Bridge	22,300	6,700 cfs	4
Cal Train Bridge	27,200	Exceeding 8,800 cfs	7
El Camino Real Bridge	27,600	Exceeding 8,800 cfs	7
San Mateo Dr Bridge	32,000	Exceeding 8,800 cfs	7
San Hill Road Bridge	38,100	Exceeding 8,800 cfs	7
Junipero Serra Blvd Bridge	40,000	Exceeding 8,800 cfs	7
Piers Lane Bridge	46,600	8,200 cfs	6
Alpine Road Bridge	47,200	Exceeding 8,800 cfs	7
I-280 Bridge	49,300	Exceeding 8,800 cfs	7

Note: [1] The distance is measured along the creek centerline and is rounded to nearest 100 feet.

The results indicate that the San Francisquito Creek is generally incapable of carrying the 100-year flow for the reach downstream of the Cal Train Bridge/El Camino Real Bridge. The Cal Train Bridge is approximately 400 feet downstream of the El Camino Real Bridge. The flow capacity is only approximately 4,400 cfs for the reach downstream of and adjacent to the Highway 101 Bridge, and 5,200 cfs for the reach adjacent to the Pope/Chaucer Street Bridge. The flow capacity of the reach upstream of the Cal Train Bridge/El Camino Real Bridge, however, is capable of carrying the 100-year flow except for a small segment adjacent to the Pier Lane Bridge.

The flow capacities of the bridges show significant difference, as presented in Table 6-2. The bridges downstream of the Cal Train Bridge/El Camino Real Bridge are incapable of carrying the 100-year flood event. The flow capacities of these bridges range from approximately 4,700 cfs for the US Highway 101 Bridge, 4,900 cfs for the Pope/Chaucer Street Bridge, 6,500 cfs for the Newell Road Bridge, 6,700 cfs for the Middlefield Road Bridge, to 6,800 cfs for the University Avenue Bridge. The flow capacities of the bridges upstream of the Cal Train Bridge/El Camino Real Bridge exceed the 100-year flow except for the Pier Lane Bridge.

7 SENSITIVITY ANALYSIS

The Manning's roughness coefficients used in the flow capacity analysis were calibrated and verified using the three historic flood event and are consistent with the normal values recommended by the HEC-RAS User's Manual (USACE, 2008). However, a sensitivity analysis was conducted to test how the Manning's roughness values impact the channel capacity. A larger roughness coefficient would result in slower flow velocities and higher water surface elevations along the creek, reducing the creek's flow capacity. On the other hand, a smaller roughness value would increase the flow capacity.

The roughness values of a channel will change with vegetation conditions, bottom material, channel sedimentation/erosion and other factors. The normal ranges of the Manning's roughness coefficients for the types of channels similar to San Francisquito Creek are discussed in Section 4.

The sensitivity analysis was conducted for two cases: (1) increasing the calibrated Manning's roughness values by 0.01, and (2) decreasing the calibrated roughness values by 0.005. The test with increased roughness values represents the dense vegetation and hydraulically rougher channel conditions, which would yield a more conservative (smaller) estimate of the flow capacity. The test with decreased roughness values represents the less vegetation, clean and hydraulically smoother channel conditions, which would yield a larger flow capacity.

The flow capacities predicted with the larger and smaller roughness values are summarized in Table 7-1 for the creek, and in Table 7-2 for the bridges. The flow capacities with the calibrated roughness values are also listed in these tables. The estimated flow capacities are sensitive to the Manning's roughness values: larger roughness values will yield lower flow capacity, and vice verse. However, variation of the roughness values within the reasonable range does not change the general conclusion about the flow capacity as presented in Section 6: (1) The creek and bridges downstream of the Cal Train Bridge/El Camino Real Bridge are incapable of carrying the 100-year flow; and (2) The flow capacities of the creek and bridges upstream of the Cal Train Bridge/El Camino Real Bridge exceed the 100-year flood event except for the Pier Lane Bridge and adjacent area.

Table 7-1. Variation Range of Flow Capacities for San Francisquito Creek

Locations	Calibrated Model Capacity (cfs)	Variation Range (cfs)	
		n+0.01	n-0.005
Adjacent to US 101 Bridge	4,400	3,600	4,800
Adjacent to Newell Road Bridge	6,200	5,200	6,700
Adjacent to University Avenue Bridge	5,800	5,100	6,300
Adjacent to Pope/Chaucer St Bridge	5,200	4,400	5,700
Adjacent to Middlefield Road Bridge	6,800	5,900	7,200
Adjacent to Piers Lane Bridge	8,200	7,400	8,200
Upstream of Cal Train Bridge except for the portion adjacent to Piers Lane Bridge	Exceeding 8,800 cfs		

Note: "Adjacent" means the reach within hundreds feet upstream and downstream of the bridge.

Table 7-2. Variation Ranges of Flow Capacities for Bridges

Locations	Calibrated Model Capacity (cfs)	Variation Range (cfs)	
		n+0.01	n-0.005
US 101 Bridge	4,700	3,900	5,200
Newell Road Bridge	6,500	6,400	6,800
University Avenue Bridge	6,800	6,000	7,100
Pope/Chaucer St Bridge	4,900	4,200	5,400
Middlefield Road Bridge	6,700	5,700	7,000
Cal Train Bridge	Exceeding 8,800		
El Camino Real Bridge	Exceeding 8,800		
San Mateo Dr Bridge	Exceeding 8,800		
San Hill Road Bridge	Exceeding 8,800		
Junipero Serra Blvd Bridge	Exceeding 8,800		
Piers Lane Bridge	8,200	8,000	8,200
Alpine Road Bridge	Exceeding 8,800 cfs		
I-280 Bridge	Exceeding 8,800 cfs		

8 SUMMARY

The following summarizes the study that has been conducted and our conclusions presented in this report:

1. A one-dimensional HEC-RAS model was developed for San Francisquito Creek for the purpose of evaluating the flow capacities of the existing channel and the major bridges.

2. The geometric data of the model was developed based on the channel topography surveyed and the LiDAR data for the project area. A Digital Terrain Model (DTM) was developed by merging the field channel survey results with the LiDAR data. The HEC-GeoRAS program was then used to create the channel geometric data for import into HEC-RAS, where the geometric data was refined and completed.

3. The HEC-RAS model covers San Francisquito Creek from the creek mouth to approximately one mile upstream of Highway 280, with a total length of approximately 55,000 feet. Thirteen bridges and ten pedestrian bridges, which were identified in the field survey, were all incorporated in the HEC-RAS model.

4. The model was calibrated and verified using three historic flood events that occurred on February 13, 2000, December 16, 2002, and January 1, 2006, respectively. The model results show reasonable agreement with the field data. The calibrated Manning's roughness values for the main channel vary from 0.03 for the reach downstream of Station 87+30, 0.04 for the reach between Station 87+30 and the Newell Road Bridge, and 0.043 for the reach upstream of the Newell Road Bridge. The floodplain roughness coefficients are larger than the main channel by 0.02.

5. Four additional historical flood events, which have caused significant flooding along the creek, were modeled using the existing channel geometry. The predicted flooding locations are generally consistent with the field observations.

6. The existing flow capacities of the creek and bridges were evaluated based on the model simulations for a series of 50 synthetic flow discharges with the MHHW at the creek mouth. A sensitivity analysis was also conducted by increasing the calibrated Manning's roughness values by 0.01 and decreasing by 0.005. The purpose of the sensitivity analysis is to test how the Manning's roughness coefficients impact the

predicted flow capacities. The flow capacities and their variation ranges are summarized in Table 7-1 for the creek, and in Table 7-2 for the bridges.

7. The lower reach (downstream of the Cal Train Bridge/El Camino Real Bridge) of San Francisquito Creek is incapable of carrying the 100-year flow. The flow capacity ranges between 3,600 cfs and 4,800 cfs with a normal value of 4,400 cfs for the reach downstream of and adjacent to the Highway 101 Bridge, and between 4,400 cfs and 5,700 cfs with a normal value of 5,200 cfs for the reach adjacent to the Pope/Chaucer Street Bridge.

8. The bridges on the creek that are downstream of the Cal Train Bridge/El Camino Real Bridge are incapable of carrying the 100-year flow. The flow capacity ranges between 3,900 cfs and 5,200 cfs with a normal value of 4,700 cfs for the Highway 101 Bridge, and between 4,200 cfs and 5,400 cfs with a normal value of 4,900 for the Pope/Chaucer Street Bridge.

9. The flow capacities of the creek and the bridges upstream of the Cal Train Bridge/El Camino Real Bridge exceed the 100-year flood event except for the Pier Lane Bridge and adjacent area.

9 REFERENCES

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