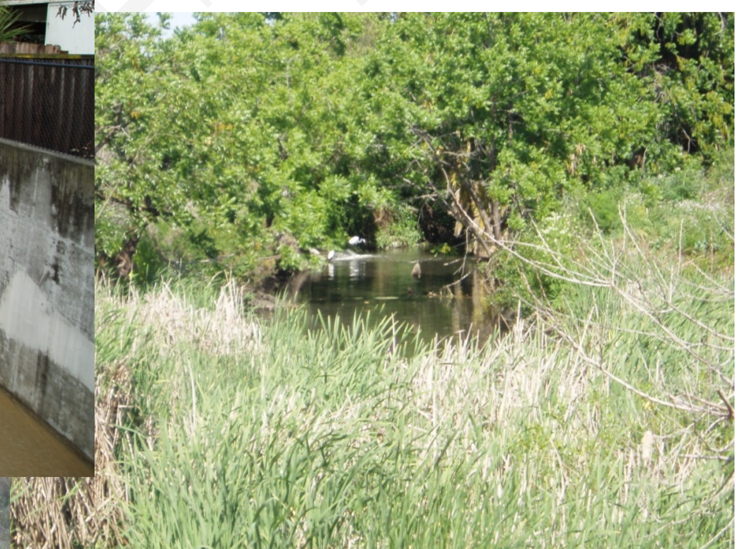
Appendix D1 – Annex 1

Coastal Engineering and Riverine Hydraulics Summary

Riverine Hydraulic Analysis for South San Francisco Bay Shoreline Study, Santa Clara & Alameda Counties, California



U.S. Army Engineer District,
San Francisco
Corps of Engineers
San Francisco, California
May 2013

South San Francisco Bay Shoreline Study
Santa Clara County & Alameda County, California

RIVERINE HYDRAULICS

U.S. Army Engineer District, San Francisco
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Executive Summary

The effects of riverine flows on the tidally-dominated South San Francisco Bay shoreline were evaluated. Additionally, the effects of the proposed coastal flood protection levee on riverine hydraulics were quantified. The extents of the tidal influence along the streams are limited to the lower reaches. The extent of the tidal influence is dependent on the relative magnitudes of the flows in the streams and the tide itself. Frequent, low-magnitude flows are much more greatly influenced by tides than less frequent, large-magnitude events like the 1% annual chance exceedance (100-year) event. For example, water surface elevations in the Guadalupe River during a 1% event will be the same upstream of Highway 237, 1000 feet from the bay, if the tide elevation is 2 ft as they would be if the tide were to be 11 ft. On the other hand, water surface elevations during a 50% annual chance exceedance (2-year) event will measurably higher (greater than 0.1 ft) as far upstream as Trimble Road (about 4 miles) over the same range of tide conditions.

The San Francisco Bay shoreline has been historically protected from the tidal influence in the south the former Cargill salt ponds and non-engineered salt pond embankments. Future plans for these former salt ponds vary and historical levels of maintenance may not continue, potentially increasing the flood risk along the San Francisco Bay shoreline and its creeks and rivers.

Without-Project Hydraulics

As part of the overall technical approach, riverine hydraulic models were developed to define the riverine response to the downstream tidal conditions within the project area. The numerical models used in this analysis include, HEC-1, HEC-RAS and FLO-2D. This study also investigated the coincidence of tide and stage for the study streams. Areas of inundation for frequency-based storm events were determined by modeling breakout flows on the floodplain.

The streams included as part of the without-project riverine analysis were: Adobe Creek, Barron Creek, Matadero Creek, Permanente Creek, Stevens Creek, Sunnyvale West Channel, Sunnyvale East Channel, Calabazas Creek, San Tomas Aquino Creek, Guadalupe River, Lower Penitencia Creek, Coyote Creek, Fremont Flood Control Channel (Line B), Scott Creek (Line A), Laguna Creek (Line E) and Aqua Caliente Creek (Line F). All of the above streams drain into tidal sloughs which connect to the Bay. An overview of the without-project study area is presented in Figure 1.

The hydrology used in this study was obtained from several sources: the Santa Clara Valley Water District (SCVWD), Alameda County Flood Control and Water Conservation

District (ACFCWCD), and the US Army Corps of Engineers, San Francisco District (USACE). The SCVWD and ACFCWCD provided the hydrology data for most of the watersheds, except for the Guadalupe River, Coyote Creek and Lower Penitencia Creek. The latter three studies were conducted by the USACE. No major land use changes are expected in the study area through Year 50 (2067) and therefore, the Year 0 to Year 50 hydrology was unchanged. Following the guidance of USACE Engineering Circular (EC) 1165-2-211, three sea level rise projections through the year 2067 were simulated: (1) a continuation of the historically observed rate (0.33 ft); (2) the National Research Council (NRC) Curve 1 (0.73 ft); and (3) NRC Curve 3 (2.13 ft).

Steady-flow HEC-RAS models were both obtained from the local sponsors and also developed independently by the USACE. For models developed by others, the USACE: updated the channel geometry, corrected bridge and culvert geometries, performed vertical datum conversions, improved estimates of Manning's n-values, geo-referenced each model and checked each model for completeness and accuracy.

Once each of the model geometries was considered complete, a calibration/validation and sensitivity analysis was conducted. Various high water marks, gage data and observed data were available for calibration and verification for six creeks, Calabazas Creek, Coyote Creek, Guadalupe River, Matadero Creek, Stevens Creek and Sunnyvale East, in Santa Clara County. For each of these streams, the steady flow hydraulic model was both calibrated by adjusting the Manning's n-value to attain reasonable agreement between the measured data and the simulated water surface profile. Model verification was completed for each of the calibrated creeks. In general, the predicted water surface elevations show good agreement with the observed data for all of the model runs. Sensitivity analyses were performed for the ungaged creeks.

Steady flow HEC-RAS models that were created for each creek were modified to analyze unsteady (time-varying) flow conditions. This allowed for the determination of the outflow hydrographs at each breakout location, and therefore a more precise estimate of flood timing and outflow volume than possible with steady flow modeling.

Levees and channel banks were modeled as lateral weirs with flood waters passing over them and spilling onto the floodplain. Unsteady HEC-RAS calculates the hydrograph of the flow passing over the structure. The locations and geometries of these structures were determined by: 1) running the 0.2% annual chance exceedance (500-year) event steady flow HEC-RAS models, (2) reviewing the longitudinal water surface and channel bank profile plots, (3) reviewing the cross section plots, (4) identifying areas where the maximum water surface elevation exceeded the bank elevation, and (5) recording the channel bank elevation and station data along the reach where capacity is likely to be exceeded. Lateral flow hydrographs calculated for each breakout point were exported from HEC-RAS into Excel. The total outflow volumes for each breakout point were calculated by integrating the incremental flow rate over the duration of flow event.

Riverine HEC-RAS modeling determined the water surface elevations within the creeks and rivers. This included defining the locations along the creeks where overtopping of lateral riverine levees occurred. From each of the riverine models, the water volumes leaving the channel in the coastal zone were used as input to the uncertainty analysis and the riverine floodplain inundation modeling.

A coincident frequency analysis was performed to determine the effects of coincidence of the peak tide and peak stream discharge and to determine the downstream boundary water surface levels. The coincident frequency analysis predicted the downstream boundary condition, influenced by tidal stage. The coincident frequency analysis developed a probability for the riverine downstream boundary condition using the method of total probability. Hourly tide probability distribution functions at the river mouths were obtained in the vicinity of Adobe Creek, Permanente Creek, Stevens Creek, Guadalupe Slough, Alviso Slough and Coyote Creek. The probability distribution functions for these six locations were then discretized to represent a portion of the range and probability. The probability was divided into ranges and an average of the values in each range was chosen, which then took on the probability of the range. The year 50 probability distribution function takes into account sea level rise by adding a constant of 0.72 feet to each of the probability curves.

Breakout hydrographs from the unsteady HEC-RAS models were used to model floodplain inundation using FLO-2D (FLO-2D Software Inc 2009). Overland flow was simulated in two dimensions based on the land surface topography as contained in the digital terrain model (DTM) and the hydrologic inputs. Due to averaging of surface topography within model grid cells (30x30 feet or 40x40 feet square elements), there could be areas where narrow dikes and roads were smoothed or erased in the models. The only additional feature which was included in the models was a 3.5 feet high concrete middle barrier running along portions of Highways 101, 237, and 880. The models were used to route flood flows from the breakout locations through the urban areas as they progressed towards the San Francisco Bay. Flood inundation maps were created for each of the frequency based storm events. The flood inundation maps were mapped in GIS based on the depths predicted by the FLO-2D models.

Floodplain FLO-2D models were not calibrated due to the absence of relevant calibration data. Sensitivity analyses were performed on a range of modeling parameters to evaluate the reasonableness of the model results. The parameters tested included: (1) Manning's roughness coefficient, (2) Area Reduction Factors, and (3) model grid cell size. Sensitivity analyses consisted of changing selected modeling parameters (while keeping other modeling parameters unchanged) and assessing the change in the simulated results.

Sensitivity analyses were conducted using the Coyote Creek west floodplain model as an example. This model had a moderate inundation area and relatively short run times, which allowed testing different modeling parameters within reasonable time limits. Variations in results due to the change in modeling parameters were assessed by comparing computed inundated areas, inundation depths, and overall flooding patterns. Given the similar topographic and hydraulic conditions, results of sensitivity tests conducted for the Coyote Creek west floodplain model are considered to be representative of the other basins modeled in this study. The sensitivity runs were conducted for the 0.2% annual chance exceedance (500-year) event. On the whole, given the complex surface topography and heavily urbanized character of the model area, differences in the modeling results caused by varying surface roughness appear to be rather insignificant.

HEC-RAS analyses found that the breakout flow rates did not change significantly from Year 0 to Year 50, even when accounting for the projected sea level rise. From the coincident frequency analysis it was found that the year 50 sea level change has little affect on the downstream boundary conditions, such that there is little change between Year 0 and Year 50 water surface elevations. Therefore, there is little to no change in the volume of water leaving the streams and entering the floodplains from Year 0 to Year 50. As a result there is no change in the riverine flood inundation maps for Year 0 and Year 50.

With-Project Hydraulics

The scope of the shoreline protection project was reduced subsequent to the without-project hydraulics analysis. As of the time of this writing, only the section of coastline between the Guadalupe River (Alviso Slough) and Coyote Creek was being considered for coastal levee construction. As a result, the with-project hydraulics analysis was limited to those two watercourses.

Proposed coastal levees will tie in to existing riverine levees and have a maximum crest elevation of 16.5 ft (NAVD 88). The proposed geometry would not reduce the available flow area or constrict the flow; therefore, it would likely not have an impact on water surface elevations in Guadalupe River/Alviso Slough or Coyote Creek.

HEC-RAS models of Coyote Creek and Guadalupe River/Alviso Slough used in the without-project analysis were modified per the proposed design. The left (south) levee crest elevation on Coyote Creek was increased from 15.81 ft to 16.5 ft NAVD 88 at River Station (RS) 74+05. The right (east) levee crest elevation was increased from 15 ft to 16.5 ft at RS 223+29.96 in the Guadalupe River model. No other changes were to the without-project Coyote Creek or Guadalupe River without-project model geometries.

Without-project coincident frequency analyses assumed that coastal water surface elevations and riverine flows are independent. Subsequent to the original study, it was shown that flow in the Guadalupe River is well-correlated with storm surge, and that tidal residuals of up to two feet may be expected due to the correlation. The maximum tidewater elevation modeled under without-project conditions was 13 feet. Maximum tidewater elevations were increased in the with-project models to 15 feet to account for storm surge effects. Minimum tidewater elevation in both without- and with-project conditions was 2.83 ft NAVD 88.

Flow hydrographs representing the 1%, 0.4% and 0.2% annual chance exceedance (100-, 250-, and 500-year) events were used for the with-project analysis. Riverine levees on both Coyote Creek and Guadalupe River were designed to safely contain the 1% annual chance exceedance (100-year) event. Flows of magnitude equal to or less than the 1% annual chance exceedance (100-year) event will be contained in the channels. Neither the modification of the cross section geometries (to account for the coastal levee) nor increasing the tidewater elevation to a maximum value of 15 ft NAVD 88 had a significant effect on predicted backwater profiles or breakout flow rates.

South San Francisco Bay Shoreline Study

Riverine Hydraulics

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SOUTH SAN FRANCISCO BAY SHORELINE STUDY

RIVERINE HYDRAULICS

1.0 PURPOSE

This report presents the river routing modeling effort for the South San Francisco Bay Shoreline Study (SSFBSS). Flooding from the river sources can exacerbate coastal flooding. The goal of this study is to examine the interaction between the creeks and rivers that flow into the South San Francisco Bay. This report focuses on the development of the hydrologic and hydraulic analyses, coincident frequency analysis and riverine floodplains for the year 0, 2017, and year 50, 2067 without project condition.

2.0 SSFBSS COORDINATE SYSTEM AND DATUM

The coordinate system reference for all models referenced in this report is the California State Plane NAD 83, Zone 3 (0403), in US Survey Feet, as the horizontal coordinate system. The vertical datum is NAVD 88. All model boundary conditions and output will be referenced to this datum.

3.0 WATERSHED DESCRIPTION

The South San Francisco Bay Shoreline study area receives water from 5 tributary watersheds, Lower Peninsula Watershed, West Valley Watershed, Guadalupe River Watershed, Coyote Creek Watershed within Santa Clara County and the Zone 6 (Agua Fria Creek Basin) tributary watershed within Alameda County for a total contributing watershed area of 673 square miles. The watershed area is bordered by the San Francisquito Creek watershed on the Peninsula, the Alameda Creek watershed in the East Bay Area, by the Santa Cruz Mountains to the west and the Diablo Range to the East. The largest contributing watershed, located in Santa Clara County, Coyote Creek, encompasses approximately 48% of the total contributing area. Elevations within the contributing watersheds range from sea level at the San Francisco Bay to 3791 feet above sea level at Loma Prieta and 3000 feet above sea level within the Diablo Range. The study watersheds are shown on Plate 1.

The valley floor once consisted of broad alluvial fans that were formed as streams emerged from the foothills, flattened, slowed and spread out, dropping out unconsolidated material. Both Santa Clara and Alameda Counties can be characterized as a primarily flat valley area adjacent to the San Francisco Bay, which have undergone rapid and extensive urbanization. The surrounding foothills have undergone minor low density urbanization, while the steep mountainous regions have remained mostly rural, open space.

In keeping up with the urbanization of the valley floor, the creek channels have also been urbanized over the years as the valley was developed for agricultural purposes in the late 1800's. The area was known as Blossom Valley, due to its abundant almond, apricot, plum, walnut, cherry and pear orchards. As suburban development grew over the years, many of the creek channels were moved, realigned and straightened. Today most of the creek channels are a combination of earthen trapezoid and concrete channels, bypasses, floodwalls and levee systems. The area is known today as Silicon Valley.

There are two US Army Corp of Engineers Flood Control Projects located on the Guadalupe River and Coyote Creek, the two largest watersheds in the SSFBSS. All other flood control projects have been completed by the Santa Clara Valley Water District (SCVWD) or the Alameda County Flood Control and Water Conservation District (ACFCWCD).

The study area is located within the valley floor portion of the five tributary watersheds which flow through the Alviso salt ponds area adjacent to the South San

Francisco Bay. The streams within these five watersheds located in Santa Clara and Alameda Counties are shown in Table 1.

Table 1. SSFBSS Watersheds

Watershed	Slough to SF Bay	Creek
Lower Peninsula	Charleston Slough	Adobe Creek Barron Creek
	Mayfield Slough	Matadero Creek
	Mountain View Slough	Permanente Creek
	Whisman Slough	Stevens Creek
West Valley	Guadalupe Slough	Sunnyvale West Channel Sunnyvale East Channel Calabazas Creek San Tomas Aquino Creek
Guadalupe	Alviso Slough	Guadalupe River
Coyote	Coyote Slough	Lower Penitencia Creek Coyote Creek
Zone 6	Coyote Slough	Fremont Flood Control Channel (Line B) Scott Ck (Line A)
	Mud Slough	Laguna Creek (Line E) Aqua Caliente Creek (Line F)

4.0 CLIMATE

The South San Francisco Bay Area experiences a mild Mediterranean climate. The San Francisco Bay strongly influences the temperatures of the areas closest to the bay, with the influence of the marine environment and on shore winds. The summers are warm, with average temperatures in the 70's and winters tend to be cool, average temperatures in the 50's in the flat valley closest to the bay. The temperatures in the Santa Cruz Mountains are more extreme, with average summer temperatures in the 80's and average winter temperatures in the 30's. Typically, most of the rainfall occurs during the winter months, approximately 80% of the area's rainfall occurs between November and March. The valley floor experiences an average annual rainfall of approximately 15 inches, while the Santa Cruz Mountains to the west and Mount Diablo to the east average approximately 50 and 24 inches of rainfall per year, respectively.

5.0 RESERVOIRS AND FLOOD CONTROL BASINS.

There are eleven reservoirs, located in the upper watersheds of Santa Clara County. SCVWD operates ten reservoirs in Santa Clara County; of these ten reservoirs eight are located in the study watersheds. The SCVWD reservoirs were constructed in the 1930's and 1950's for water conservation and water supply purposes. The SCVWD reservoirs do provide some flood protection, though they are not specifically operated to for that purpose.

Of the SCVWD nine reservoirs, five, Lexington, Vasona, Guadalupe, Almaden and Calero, are located within the Guadalupe River Watershed. Additionally, Lake Elsman is located within the Guadalupe River watershed; however, it is operated by the San Jose Water Company. The SCVWD also operates the Stevens Creek dam and reservoir within the West Valley Watershed and Coyote Dam and Anderson Dam and Reservoirs within the Coyote Watershed.

The Palo Alto Flood Basin (PAFB) is located within the Lower Peninsula Watershed and Palo Alto Baylands, adjacent to the San Francisco Bay and receives inflow from three creeks within the West Valley Watershed, Matadero, Barron and Adobe Creeks. The PAFB was built in 1956 with the construction of levees, enclosing a 600-acre tidal marsh area. Unlike the reservoirs within the study watersheds, the primary purpose of the PAFB is flood control. Inflow to the flood basin is stored until the stage in the PAFB is higher than the stage of the bay. The PAFB is owned by the City of Palo Alto, however the SCVWD has maintenance easements for the levees and operates 15 of the 16 tide gates. The City of Palo Alto is responsible for one of the 16 tide gates.

There are no reservoirs within the Alameda County study area.

6.0 RIVERINE STUDY LIMITS

The study limits for each stream is shown in Table 2. The downstream limits are at the mouth of the stream with the bay, Mud Slough or Coyote Creek. Except Adobe, Barron and Matadero Creeks, the downstream limits are the Palo Alto Flood Basin. The extents of the riverine study limits are shown on **Plate 2.**

Table 2. SSFBSS Riverine Study Limits

Creek	HEC-RAS Upstream Study Limits
Adobe	El Camino Real
Aqua Caliente, Zone 6 Line F	NUMMI Bridge
Barron	El Camino Real
Calabazas	El Camino Real
Coyote	Montague Expressway
Fremont Flood Control Channel, Zone 6 Line B	Warm Springs Rd
Guadalupe River/Alviso Slough	Highway 880
Guadalupe Slough	Calabazas Creek
Laguna Creek, Zone 6 Line E	Grimmer Road
Lower Penitencia	Marilyn Drive
Matadero	El Camino Real
Permanente	Villa Street
Stevens	Central Expressway
Scott	Warm Springs Blvd
Sunnyvale East	Hwy 101
Sunnyvale West	Hwy 101
San Tomas Aquino	Hwy 101

7.0 BASIS OF HYDROLOGY

The expected annual exceedance probability is necessary in determining the project performance. The expected annual exceedance probability is the probability that the specified discharge will be exceeded in any given year. The exceedance probabilities or peak discharges for this study were obtained from several sources, the SCVWD, ACFCWCD and the San Francisco District (District). The SCVWD and ACFCWCD provided the hydrology data for most of the watersheds, except for the Guadalupe River, Coyote Creek and Lower Penitencia Creek.

Both the SCVWD and ACFCWCD conducted flood-runoff analyses using HEC-1, the US Army Corps of Engineers, Flood Hydrograph Package. HEC-1 is a single event rainfall-runoff model which estimates stream runoff time series data based on precipitation data input by the user. HEC-1 is a US Army Corps of Engineers legacy software package, developed by the Hydrologic Engineering Center that is still widely used by consulting firms and public agencies today. The SCVWD and ACFCWCD HEC-1 models depict the response of individual watersheds for storm events of various expected annual exceedance probabilities.

The SCVWD models reflect the expected watershed conditions up to year 2067, year 50. This also includes year 0, 2017, hydrology for this study. No major land use changes are expected in the Lower Peninsula and West Valley Watersheds and the ACFCWCD models represent build-out watershed conditions.

Excerpts from the SCVWD and the ACFCWCD hydrology reports can be found in **Appendix A**. The peak flood discharges for the existing conditions and the year 0 (2017) SSFBSS hydrology, as reported by the local agencies, are summarized in the following tables.

Table 3a. SCVWD Peak Flood Discharges Lower Peninsula Watershed[1] (cfs)

Location	Drainage Area (sq.mi.)	Percent Chance Exceedance					
		43%	20%	10%	4%	2%	1%
Adobe Ck at El Camino Real	9	680	1200	1600	2200	2600	3000
Adobe Ck u/s Barron Ck	11.06	750	1300	1700	2300	2700	3100
Barron Ck at El Camino Real	2.48	210	220	220	220	230	230
Barron Ck at Hwy 101	3.04	280	310	320	330	340	350
Adobe-Barron d/s Hwy 101	14.1	970	1500	2000	2600	3000	3400
Matadero Ck at El Camino Real (USGS 11166000)	7.63	670	1000	1500	2000	2400	2700
Matadero Ck at Alma Street	9.75	860	1300	1700	2200	2500	2800
Matadero Ck at HWY 101	13.99	1000	1500	1900	2400	2700	3000
Permanente Ck d/s Hale Ck	13.98	300	630	970	1500	1900	2300
Permanente Ck at Alma Street	15.76	360	730	1100	1600	2100	2500
Permanente Ck at US Hwy 101	16.53	420	810	1200	1700	2200	2600
Stevens Ck at El Camino	26.49	2200	3500	4600	5900	6900	7800
Stevens Ck at US Hwy 101	29.79	2500	3800	4900	6300	7200	8100

Sources: 1/ Santa Clara Valley Water District, Lower Peninsula Watershed Hydrology, Revised December 2007.

Table 3b. SCVWD Peak Flood Discharges West Valley Watershed[2] (cfs)

Location	Drainage Area (sq.mi.)	Percent Chance Exceedance					
		43%	20%	10%	4%	2%	1%
Sunnyvale West at Maude Ave	2.05	200	240	260	280	300	310
Sunnyvale West d/s Hwy 237	2.81	250	290	320	350	360	380
Sunnyvale East d/s Hwy 101	6.34	570	700	780	870	930	980
Sunnyvale East d/s Hwy 237	7.09	650	790	880	980	1000	1100
Sunnyvale East d/s Caribbean Dr	7.25	680	810	900	990	1000	1100
Calabazas Ck u/s El Camino Storm Drain	14.27	1200	1700	2100	2600	2900	3150
Calabazas Ck at SPRR / d/s el camino	17.06	1400	2000	2400	2900	3300	3600
Calabazas Ck D/s Higway 237 pump station	21.14	1600	2200	2650	3200	3600	3900
San Tomas Aquino Ck at Hwy 101	42.23	3200	4600	5800	7200	8100	9000
San Tomas Aqunio Ck at Hwy 237	44.9	3300	4800	6000	7400	8300	9200

Sources:2/ Santa Clara Valley Water District, West Valley Watershed Hydrology Report, January 2008.

Table 3c. ACFCWCD Peak Flood Discharges Zone 6 [3,4,,5] (cfs)

Location	Drainage Area (sq.mi.)	Percent Chance Exceedance				
		50%	10%	2%	1%	.2%
Fremont Flood Control Channel (Line B) Warm Spings Blvd	1.03	-	200	380	455	600
Fremont Flood Control Channel (Line B) Confluence with B-2 at SPRR	1.34	-	230	465	560	760
Fremont Flood Control Channel (Line B) Interstate 880	1.57	-	280	555	670	910
Scott Creek at Warm Springs	0.9	75	299	570	675	991
Scott Creek at SVRT Tracks	1.4	77	310	592	700	1028
Scott Creek at I-880	1.8	90	363	693	820	1204
Laguna Ck u/s Auto Mall Parkway (ED)C	15.9	227	983	1725	1974	-
Laguna Ck u/s maintenance bridge (EE)C	17.29	291	1249	2005	2287	-
Laguna Ck (EF)C	18.94	344	1570	2353	2696	-
Laguna Ck u/s I880 (EG)C	20.87	479	1810	2611	2937	-
Laguna Ck u/s Cushing Pkwy (EH)C	21.78	538	1855	2695	3019	-
Laguna Ck d/s confluence with Agua Calinente (EI)C	24.56	599	2229	3530	3973	-
Agua Caliente NUMMI Bridge (FD)C	2.47	63	495	904	1064	-
Agua Caliente At Confluence with Laguna Ck (FC)C	2.63	71	506	930	1090	-

Sources: 3/ Laguna Creek (Zone 6, Line E) Flood Control Project Alternatives Analysis and Feasibility Study, April 2003. 4/ Watershed Studies Program Hydrologic Study Report for Zone 6, Line B, ACFCWCD. 5/ HEC-1 models obtained from ACFCWCD, January 2008

The hydrology for the US Army Corps of Engineers flood control projects, the Guadalupe River and Coyote Creek, are based on the District's *1977 Hydrologic Engineering Office Report, Guadalupe River and Coyote Creek, Santa Clara County, California.* This is the same hydrologic analysis used in the General Design Memorandums for both completed flood control projects. The 1977 results are estimated for the year 2010 conditions for both the Guadalupe River and Coyote Creek.

In November 2009 the District completed the *Guadalupe Watershed Hydrologic Assessment*. The 2009 study updated the study methodology and results of the 1977 hydrology. The 2009 study results were found to be similar to the 1977 report. The peak discharge at the San Jose gage (USGS gage #11169000) for the 1% chance exceedance event was estimated at 17,967 cfs in 2009 and 17,000 cfs in the 1977 report, a 6% difference. The 2009 results are estimated for full built-out conditions. Since the difference in flow rates from 1977 to 2009 are so small, less than 10%, the changes in flow are not expected to change the results of the Guadalupe River hydraulic model for this study. However, the 2009 study results will be incorporated as this study moves forward.

The Lower Penitencia Creek is currently part of the Corps of Engineers, Sacramento District (SPK) and SCVWD Berryessa Creek Levees Project. The hydrology for Lower Penitencia Creek is included in the *Berryessa Creek Watershed Hydrology Report, October 2006*, completed by Northwest Hydraulic Consultants for SCVWD and SPK.

The peak flood discharges for the Guadalupe River, Coyote Creek and Lower Penitencia Creek are shown in Table 4. The hydrologic analyses reflect build-out conditions for each of the watersheds. Each of the reports is available for review at the District's office.

Table 4. Guadalupe River, Coyote Creek and Lower Penitencia Creek Peak Discharges (cfs)

Location	Drainage Area (sq.mi.)	Percent Chance Exceedance							
		50%	20%	10%	4%	2%	1%	.4%	.2%
Guadalupe Rv at San Jose[1]	144	2,700	4,500	6,700	9,700	13,500	17,000	21,000	32,000
Guadalupe Rv at San Jose[2]	145.6	3,317	6,059	7,712	10,463	14,251	17,967	22,431	27,942
Coyote Ck at Hwy 237[1]	320.89	3,300	6,200	8,400	10,500	13,000	14,500	16000	18000
Lower Penitencia at Coyote Creek[3]	29.1	2,480	3,640	4,310	5,900	6,980	8,720	10,790	12,080

Sources: 1/_1977 Hydrologic Engineering Office Report, Guadalupe River and Coyote Creek, Santa Clara County, California. 2/ Guadalupe Watershed Hydrologic Assessmen, USACE-SPN 2009. 3/ Berryessa Creek Watershed Hydrology Report, October 2006.

The Corps of Engineers requires the use of risk-based analysis procedures for formulating and evaluating flood damage reduction measures. Corps of Engineers planning studies are evaluated for performance against a range of events, including events that exceed the capacity. The eight peak discharges used in this study are the 50-, 20-, 10-, 4-, 2-, 1-, 0.4- and 0.2-percent events.

The peak discharges provided by the SCVWD and ACFCWCD, shown in Tables 2a, 2b and 2c, are not entirely consistent with the default eight peak discharges commonly used by the Corps of Engineers. To get the various peak discharges not included in the ACFCWCD and SCVWD analyses, the Exceedance Probability Functions with Uncertainty, the analytical discharge-probability function in the Hydrologic Engineering Center's Flood Damage Analysis software package (HEC-FDA) was used.

The equivalent length of record was determined using the guidelines described in EM 1110-2-1619, USACE Risk Based Analysis for Flood Damage Reduction. The SCVWD hydrology models are calibrated to several events. Per the guidelines an equivalent length of record (N) of 10-30 years should be used. The guidelines state that the equivalent record length is based on "J*udgment to account of the quality of any data used in the analysis, for the degree of confidence in models and for previous experience with similar studies."* An equivalent length of record was estimated to be 25 years for all analyses.

The length of record was used along with the peak flows to calculate confidence limits for the discharge frequency curve using HEC-FDA. A graphical method was used to generate the error bands surrounding the 50-, 20-, 10- , 4-, 2-, 1-, 0.4- and 0.2-percent events. The error band for the higher frequency events was interpolated using the analytical method in HEC-FDA to estimate the discharge for the 99.9-percent event. The results of these analyses for the most downstream point along each stream are presented in Table 5 and on **Plates 2-17**.

The error bands are not plotted for the Corps of Engineers projects. The Coyote Creek and Guadalupe River project were completed prior to risk-based procedures and the Lower Penitencia Project is part of a Sacramento District project.

Table 5. Peak Flows for Santa Clara County and Alameda County Study Reaches (Assumes All Flow Contained in Channel) (cfs)

Location	Drainage Area (sq.mi.)	Percent Chance Exceedance							
		50%	20%	10%	4%	2%	1%	.4%	.2%
Adobe Ck at El Camino Real	9	580	1200	1600	2200	2600	3000	3540	3945
Adobe Ck u/s Barron Ck	11.06	640	1300	1700	2300	2700	3100	3910	4020
Barron Ck at El Camino Real	2.48	210	220	220	220	230	230	235	240
Barron Ck at Hwy 101	3.04	275	310	320	330	340	350	360	365
Adobe-Barron d/s Hwy 101	14.1	850	1500	2000	2600	3000	3400	3910	4285
Matadero Ck at El Camino Real (USGS 11166000)	7.63	610	1000	1500	2000	2400	2700	3000	3300
Matadero Ck at Alma Street	9.75	680	1300	1700	2200	2500	2800	3100	3400
Matadero Ck at HWY 101	13.99	990	1500	1900	2400	2700	3000	3480	3760
Permanente Ck d/s Hale Ck	13.98	240	630	970	1500	1900	2300	2885	3350
Permanente Ck at Alma Street	15.76	295	730	1100	1600	2100	2500	3100	3580
Permanente Ck at US Hwy 101	16.53	350	810	1200	1700	2200	2600	3190	3660
Stevens Ck at El Camino	26.49	1900	3500	4600	5900	6900	7800	8800	9700
Stevens Ck at US Hwy 101	29.79	2200	3800	4900	6300	7200	8100	9260	10100
Sunnyvale West at Maude Ave	2.05	190	240	260	280	300	310	320	330
Sunnyvale West d/s Hwy 237	2.81	240	290	320	350	360	380	400	410
Sunnyvale East d/s Hwy 101	6.34	550	700	780	870	930	980	1000	1100
Sunnyvale East d/s Hwy 237	7.09	610	790	880	980	1000	1100	1150	1200
Sunnyvale East d/s Caribbean Dr	7.25	660	810	900	990	1000	1100	1150	1200
Calabazas Ck u/s El Camino Storm Drain	14.27	1070	1700	2100	2600	2900	3150	3500	3740
Calabazas Ck at SPRR / d/s el camino	17.06	1370	2000	2400	2900	3300	3600	4060	4400
Calabazas Ck D/s Higway 237 pump station	21.14	1450	2200	2650	3200	3600	3900	4330	4640
San Tomas Aquino Ck at Hwy 101	42.23	2850	4600	5800	7200	8100	9000	10100	10900

Table 5(continued). Peak Flows for Santa Clara County and Alameda County Study Reaches (Assumes All Flow Contained in Channel) (cfs)

Location	Drainage Area (sq.mi.)	Percent Chance Exceedance							
		50%	20%	10%	4%	2%	1%	.4%	.2%
San Tomas Aqunio Ck at Hwy 237	44.9	2950	4800	6000	7400	8300	9200	10270	11030
Fremont Flood Control Channel (Line B) Warm Springs Blvd	1.03	50	130	200	300	380	455	500	600
Fremont Flood Control Channel (Line B) Confluence with B-2 at SPRR	1.34	60	145	230	360	465	560	640	760
Fremont Flood Control Channel (Line B) Interstate 880	1.57	90	190	280	410	555	670	780	910
Scott Creek at Warm Springs	0.9	75	194	299	447	570	675	745	991
Scott Creek at SVRT Tracks	1.4	77	201	310	464	592	700	773	1028
Scott Creek at I-880	1.8	91	235	363	543	693	820	906	1204
Laguna Ck u/s Auto Mall Parkway (ED)C, Line E	15.9	227	594	983	1383	1725	1974	2319	2593
Laguna Ck u/s maintenance bridge (EE)C, Line E	17.29	291	757	1249	1665	2005	2287	2676	2985
Laguna Ck (EF)C, Line E	18.94	344	932	1570	2008	2353	2696	3172	3551
Laguna Ck u/s I880 (EG)C, Line E	20.87	479	1147	1810	2261	2611	2937	3380	3727
Laguna Ck u/s Cushing Pkwy (EH)C	21.78	538	1213	1855	2328	2695	3019	3457	3799
Laguna Ck d/s confluence with Agua Calinente (EI)C	24.56	599	1420	2229	2747	3530	3973	4576	5047
Agua Caliente NUMMI Bridge (FD)C	2.47	63	244	495	714	904	1064	1293	1480
Agua Caliente At Confluence with Laguna Ck (FC)C	2.63	71	258	506	732	930	1090	1318	1503
Guadalupe Rv at San Jose	144	2,700	4,500	6,700	9,700	13,500	17,000	21,000	32,000
Coyote Ck at Hwy 237	320.89	3,300	6,200	8,400	10,500	13,000	14,500	16000	18000
Lower Penitencia at Coyote Creek	29.1	2,480	3,640	4,310	5,900	6,980	8,720	10,790	12,080

7.1 YEAR 0 & YEAR 50 HYDROLOGY

The hydrology presented in Section 5 assumes that all the flow is contained within the channel. This assumes that each creek contains the 50- thru the 0.2- percent flood events to the study limits. However, this does not represent the conditions out in the field. Where existing information was available the upstream channel capacities were taken into account and used in the year 0, 2017 hydraulic analysis. The creeks where upstream capacity restrictions affect the year 0 hydrology are presented in Table 6.

Table 6. Hydrology based on Capacity Limitations (cfs))

Location	Drainage Area (sq.mi.)	Percent Chance Exceedance							
		50%	20%	10%	4%	2%	1%	.4%	.2%
Permanente Ck d/s Hale Ck	13.98	240	630	635	635	635	635	635	635
Permanente Ck at Alma Street	15.76	295	730	765	735	835	835	850	865
Permanente Ck at US Hwy 101	16.53	350	810	865	835	935	935	940	945
San Tomas Aquino Ck at Hwy 101	42.23	2850	3200	4600	5800	7100	7100	7100	7100
San Tomas Aqunio Ck at Hwy 237	44.9	2950	3300	4800	6000	7300	7300	7300	7270
Guadalupe Rv at San Jose	144	2,700	4,500	6,700	9,700	13,500	17,000	21,000	24050
Coyote Ck at Hwy 237	320.89	3,300	6,200	8,400	10,500	13,000	14,500	16000	17000

The reduction in flow on Permanente Creek is due to capacity limitations from Park Drive to Mountain View Road Downstream of Hale Creek. The reduction in capacity is documented in the SCVWD report, *Permanente Creek Watershed Planning Study, Project Background/Problem Definition Report, February 2004*. Flow leaves the channel in this area reducing the amount of flow downstream to 635-cfs at Villa Street, the SSFBSS study limits.

On San Tomas Aquino a restriction is located at San Tomas Expressway limiting the flow to Hwy 101 to 7100-cfs. The restriction is documented in the SCVWD report, *San Tomas Creek, Planning Study, Route 237 to Highway 101, August 1995* and the

San Tomas Aquino Creek Levee Raising Project-Letter of Map Revision Request No. 00-009-071P, City of Santa Clara, Ca, Community No. 06035, January 24, 2000.

The Guadalupe River flow is lost between Los Gatos Creek and Hwy 880, 8500-cfs is lost to the left flood plain and the channel capacity at Hwy 880 will be 24,050-cfs. This is documented in the Corps of Engineers report, *Guadalupe River, General Design Memorandum, Volume 1 of 2, December 1991*.

The reduction of flow on Coyote Creek is limited to 17,000-cfs in the vicinity of Rock Springs Road. This is due to the loss of flow from the basin in the Canoas Creek area upstream. This is documented in the Corps of Engineers report, *Coyote Creek at Rock Springs Road, Review of Existing Hydrology, January 2001*.

Hydrology for future conditions was assumed not to change significantly between year 0 (2017) and year 50 (2067). According to the San Francisquito Creek hydrology Study completed by the SCVWD (2007), the changes in future flows for the 1% event only increase by approximately 1-2%, which is considered insignificant. This is mainly due to the limited capacity of the storm drain system, which is typical of the South San Francisco Bay Area. Therefore, no changes were made to the hydrology for year 50.

8.0 DEVELOPMENT OF HYDRAULIC MODELS

The Hydrologic Engineering Center's River Analysis System, (HEC-RAS) was used to develop the water surface profiles for each of the exceedance probability events. HEC-RAS is a one-dimensional steady and unsteady flow river hydraulic modeling program, developed by the US Army Corps of Engineers.

The majority of the HEC-RAS models developed for the SSFBSS are based on existing HEC-RAS and HEC-2 models obtained from the SCVWD, ACFCWCD, and Valley Transportation Authority (VTA). A total of 41 available models were reviewed by the

San Francisco District (District) and the most current hydraulic model was chosen for use in this study. The existing models adopted for use in the SSFBSS are listed in Table 7.

Table 7. Existing Hydraulic Models Adopted for SSFBSS

Creek	File Name	Model	Model Date	Model Author	Survey Date
Adobe*	Adobe_ds.prj	HEC-RAS	June 2008	SCVWD	1991 As-Builts
Agua Caliente* (Zone 6, Line F)	F06G32 ZONE 6 LINE F (EAST) HEC-II DATA.dat, F06G32 ZONE 6 LINE F (WEST) HEC-II DATA.dat	HEC-2	UNKNOWN	ACFCWCD	2003
Barron*	BARRON.DAT	HEC-2	March 1978, updated 2001?	SCVWD	1989 as-builts
Calabazas*	Calabazas.prj	HEC-RAS	2005	GS Nolte & Assoc.	1998
Coyote*	4021019.DAT, 020.DAT, 021.DAT, 022-3.DAT, 024.DAT, 025.DAT, 026.DAT	HEC-2	1990/1991, Updated by SCVWD March 2003	SCVWD / USACE	As-builts dated 1989, 1994; Surveys dated 1989-1993.
Fremont Flood* Control Channel (Zone 6, Line B)	F06G32 ZONE 6 LINE B HEC-II DATA.dat	HEC-2	UNKNOWN	ACFCWCD	2003
Guadalupe River/ Alviso Slough*	LGRPF.prj	HEC-RAS	2003	Northwest Hydraulics	1996
Laguna Creek (Zone 6, Line E)	Existing_conditions.prj	HEC-RAS	2002	URS	unknown
Matadero*	Mataderock.prj	HEC-RAS	2006	SCVWD	unknown
Permanente*	P1.prj, P2.prj, P3.prj	HEC-RAS	Unknown	SCVWD	1998 topography; missing bathymetry in tidal areas
Sunnyvale East	06192007.prj	HEC-RAS	2007	SCVWD	Aug 2006 SCVWD Survey
Sunnyvale West Moffatt Channel	06052007.prj	HEC-RAS	2007	SCVWD	Aug 2006 SCVWD Survey
Stevens	45445_06980.prj	HEC-RAS	2003	SCVWD	1982 As-builts, updated 1992
Lower Penitencia*	4033005.DAT, 4033006.DAT	HEC-2	Nov 1990	SCVWD	1984 & 1989 as-builts
Scott Creek	LineAFinal.prj Geometry: Opt. 2	HEC-RAS	2009	AECOM	SVRT Design
Scott Creek	Prologisfinal.prj	HEC-RAS	2008	Schaaf & Wheeler	CalTrans as-builts and weir design downstream of I-880

* Model not georeferenced.

9.0 HYDRAULIC MODEL REVISIONS

Modifications to the models listed in Table 7 were completed to update the model geometry and are based on the most accurate data available at this time. The geometries based on these revisions are adequate for calculating channel capacities and floodplains. These revisions are described in the following sections. However, it should be noted, that due to the inaccuracies of the revisions, such as datum adjustments, undocumented geometry data and georeferencing, to name a few, it is recommended that the geometry for all of the following creeks be resurveyed for design purposes.

9.1 GEOREFERENCING HEC-RAS MODELS.

Georeferencing is the process in which real world coordinates are assigned to an HEC-RAS model. With the exception of the Sunnyvale East, Sunnyvale West/Moffett Channels, San Tomas Aquino, Stevens Creek and Laguna Creek, none of the other SCVWD and ACFCWCD HEC-RAS/HEC-2 models listed in Table 6 were spatially represented. To be consistent with the entirety of the SSFBSS it was necessary to georeference the Adobe Ck, Aqua Caliente, Barron Creek, Calabazas Creek, Coyote Creek, Fremont Flood Control Channel, Guadalupe River, Matadero Creek, Lower Penitencia Creek and Permanente Creek HEC-2/HEC-RAS models.

The first step to this process was to convert all the HEC-2 models to HEC-RAS version 4.0, which has the built-in capability for georeferencing. Georeferenced aerial photos were loaded into the geometric data editor in HEC-RAS. The 2003 USGS Color 1 Foot tiff images and the City of San Jose 2004 IKONOS aerial photos were referenced as the background dataset. The background aerial photo was then used to draw in the stream centerline. Landmarks such as bridges, levees, flood walls, in-stream structures and roads included in the model were then identified in the background photo. Then the GIS Tools in the Geometric Data Editor were used to edit, modify and assign x- and y-coordinates to the river network and cross sections based on the coordinate system and landmarks of the aerial photos.

9.2 MODIFICATIONS TO EXISTING HEC-RAS MODELS.

Several input parameters were checked in each model, these include cross sections, bridge/culvert dimensions, reach lengths, the direction of cross section, Manning's n-values, expansion and contraction coefficients, units and vertical datum.

However, there were a few reaches where the existing model data was severely outdated or a newer survey had become available. For these reaches the models were developed by the District. A summary of the model reaches developed for the SSFBSS by the District are summarized in Table 8.

Table 8. HEC-RAS Models Developed by San Francisco District

Creek	Reach	Model	Model Date	Model Author	Survey Date
Alviso Slough	NHCRR to Bay	HEC-RAS	2007	San Francisco District	2004/2005
San Tomas Aquino	Hwy 101 to Guadalupe Slough	HEC-RAS	2007	San Francisco District	2007
Stevens Ck	Hwy 101 to Bay	HEC-RAS	2007	San Francisco District	2007

The model geometry for these reaches was created using the HEC-GeoRAS extension in ArcMap which creates a spatially represented stream alignment and geometry using a digital elevation model (DEM). The data was then directly imported into HEC-RAS. Bridges and culverts were input based on the survey data and site visits.

9.2.1 BRIDGES/CULVERTS AND REACH LENGTHS.

Since the majority of the existing SCVWD models are 10 or more years old, many of the landmarks, particularly bridges were newer than the date of the last model update. Some bridges had been modified, since the many of the models were developed and these modifications were not accounted for in the models. Additional problems such as incorrect reach lengths, missing bridges and outdated geometry were found with various models as the models were georeferenced. Therefore site visits, new surveys, as-builts, aerial photographs and other resources were used to represent these hydraulic features as accurately as possible.

9.2.2 MANNING'S N-VALUES.

For the existing models a general review of the roughness coefficients was conducted for all creek reaches and checked for reasonableness. Overall, the Manning's coefficients were considered reasonable for the various channel surface types in the model including concrete and earthen channels with various degrees of vegetation.

For the model reaches developed by the District, Stevens Creek (downstream Hwy 101), San Tomas Aquino and Guadalupe Slough to Calabazas Creek and Alviso Slough (NHCRR Bridge to Bay), the Manning's n-values were estimated based on experience or estimated using the equation from *Chow, Open Channel Hydraulics* (1959) and expressed here:

$$n = (n_0 + n_1 + n_2 + n_3 + n_4) * m_5 \quad \text{(equation 9.1)}$$

Where:

n_0 = a basic n value for a straight, uniform, smooth channel,

n_1 = a value added to n_0 to correct for the effect of surface irregularities,

n_2 = a value for variations in shape and size of the channel cross section,

n_3 = a value for obstructions,

n_4 = a value for vegetation and flow conditions, and

m_5 = a correction factor for meandering of channel.

In general, where possible the Manning's n-values were verified from site visits, as-builts, and the SCVWD "yellow book". Assumptions made regarding channel characteristics when selecting appropriate Manning's n values are presented in Table 9.

Table 9. – Assumptions on Channel Characteristics for Manning's n Selection

Manning's n Value	Description
0.04	Low to medium amount of moderately dense vegetation along the banks, no significant amount of vegetation along the channel bottom. Also representative of floodplain area in agricultural regions.
0.05	Medium amount of brushy, moderately dense vegetation along the banks, no significant amount of vegetation along the channel bottom.
0.06	Medium amount of vegetation, similar to 1- to 2-year old willow trees inter-grown with weeds and brush along the benches and side slopes.
0.07	Medium amount of vegetation similar to 1- to 2-year old willow trees inter-grown with weeds and brush along the benches and side slopes.
0.1	Large amount vegetation similar 8- to 10- year old willow trees inter-grown with weeds and brush along the benches and side slopes.

9.2.3 CROSS SECTION DIRECTIONS.

From the model notes, it was unclear if all cross section data had been entered in the standard HEC-RAS station convention of numbering increasing from left to right when looking in the "downstream" direction. A few model's notes stated the cross section direction was read as looking upstream. Therefore the cross section direction in each model was verified and corrected if necessary.

A combination of aerial photographs (USGS 1' tiff images) and site visits were used to check cross section orientation. Locations showing floodwalls, access roads and levees were identified and checked against cross section geometry. If necessary, the cross section was corrected in HEC-RAS by "flipping" the cross section to "left to right looking downstream". The initial orientation of the cross sections in each of the adopted models is noted in Table 10. Additionally, if the cross section direction required correction it is noted in Table 10 as "flipped".

Table 10. Adopted Hydraulic Models, Vertical Datum & Cross Section Direction

Creek	**File Name**	**Model**	**Cross Section Direction**	**Vertical Datum**	Units
Adobe	Adobe_ds.prj	HEC-RAS	L-R lkg u/s - flipped	NGVD29	Feet
Agua Caliente (Zone 6, Line F)	F06G32 ZONE 6 LINE F (EAST) HEC-II DATA.dat, F06G32 ZONE 6 LINE F (WEST) HEC-II DATA.dat	HEC-2	L-R lkg u/s - flipped	NAVD88	Feet
Barron	Barron.DAT	HEC-2	L-R lkg u/s - flipped	NGVD29	Feet
Calabazas	Calabazas.prj	HEC-RAS	L-R lkg d/s	NGVD29	Feet
Coyote	4021019.DAT, 020.DAT, 021.DAT, 022-3.DAT, 024.DAT, 025.DAT, 026.DAT	HEC-2	d/s Hwy 237: L-R lkg u/s - flipped & u/s Hwy 237: L-R lkg d/s	NGVD29	Feet
Fremont Flood Control Channel (Zone 6, Line B)	F06G32 ZONE 6 LINE B HEC-II DATA.dat	HEC-2	L-R lkg u/s - flipped	NAVD88	Feet
Guadalupe River	LGRPF.prj	HEC-RAS	L-R lkg d/s	NAVD88	Meters
Laguna Creek* (Zone 6, Line E)	Existing_conditions.prj	HEC-RAS	L-R lkg d/s	NAVD88	Feet
Matadero	Mataderock.prj	HEC-2	L-R lkg d/s	NGVD29	Feet
Permanente	P1.prj, P2.prj, P3.prj	HEC-RAS	L-R lkg d/s	NAVD88	Meters
Scott	Prologisfinal.prj	HEC-RAS	L-R lkg d/s	NAVD88	feet
Sunnyvale East	06192007.prj	HEC-RAS	L-R lkg d/s	NAVD88	Feet
Sunnyvale West / Moffatt Channel	06052007.prj	HEC-RAS	L-R lkg d/s	NAVD88	Feet
Lower Penitencia	4033005.DAT, 4033006.DAT	HEC-2	L-R lkg d/s	NGVD29	Feet
Stevens	45445_06980.prj	HEC-RAS	L-R lkg u/s - flipped	NGVD29	Feet

Notes: L-Left, R-Right, lkg-Looking, u/s-upstream, d/s-downstream

9.2.4 VERTICAL DATUM AND UNITS.

The SCVWD models used both NGVD29 and NAVD88 and units of feet and meters. However, not every model stated which datum or the units the geometry data referenced. Therefore, elevation data was cross referenced between the HEC-2 data files and other data sources, such as reports, as-builts, the USGS Bathymetry, and other topographic maps and cross sections with a known vertical datum and units. The units and vertical datums determined are shown in Table 10.

Each of the ACFCWCD hydraulic models has a vertical datum NAVD88, feet, and model cross sections were input left to right looking downstream.

9.2.5 VERTICAL DATUM CONVERSION FROM NGVD29 TO NAVD88.

The SSFBSS Project vertical datum is NAVD88 with units of feet. Therefore, if the vertical datum and units were determined to be in NGVD29 or in metric units, the appropriate conversion was performed. For converting to NAVD88, the accepted project-wide conversion from NGVD29 to NAVD88 is to add (+) 2.75 feet for this study. The true shift between the two datums should vary spatially. However, because of the inherent inaccuracies of the hydraulic models due to the lack of survey data, georeferencing errors and other assumptions made about the models it was determined that a constant conversion from NGVD29 to NAVD88 would be sufficient for this project.

9.3 PALO ALTO FLOOD BASIN.

Matadero Creek, Barron Creek and Adobe Creek all flow into the Palo Alto Flood Basin (PAFB), with a combined drainage area of 30 square miles. These three creeks flow thru the City of Palo Alto and the border of the City of Mountain View. The PAFB stores creek runoff during high tides and is closed to the Bay during high tides.

Matadero Creek flows directly into the PAFB at the northern edge of the basin into Mayfield Slough. From the upstream limits of the study area to Hwy 101 Matadero

Creek consists of a U-frame concrete or trapezoidal concrete channel with concrete floodwalls to Greer Road. *Downstream of Alma Street, Matadero Creek consists of an entirely man-made channel which historically did not exist as a natural watercourse*. (Schaaf & Wheeler) From Greer Road to Hwy 101 the channel consists of a sacked concrete side slopes with an earthen channel bottom. Downstream of Hwy 101, Matadero Creek flows through an earthen trapezoid channel with dense vegetation as it enters the PAFB.

Matadero Creek was georeferenced in the manner described in **Section 9.1**.

Additionally, geometry and cross section data improvements were made to the HEC-RAS model geometry, improved bridge section representation, corrected downstream reach lengths, levee walls added and merged the downstream bypass with the Matadero Creek model. The Matadero Creek bypass splits off along the right side around the maintenance yard, improving flow conveyance downstream of Highway 101. Matadero Creek and the bypass cross sections were merged into one cross section to appropriately incorporate data from both data sets.

Adobe Creek and Barron Creek combine upstream of Hwy 101 and flow into the basin at the southern edge of the PAFB into Charleston Slough. Barron Creek within the study reach is a concrete lined trapezoidal channel, with floodwalls downstream of Louis Road. Adobe Creek also consists of a concrete line trapezoidal and U-Frame Channel. Downstream of the Adobe/Barron confluence the channel is concrete lined. Downstream of Hwy 101 Adobe /Barron Creek flow through an earthen channel with dense vegetation as it enters the PAFB.

Because Barron Creek and Adobe Creek are part of the same system the individual HEC-2 models for Adobe and Barron Creeks were combined into one georeferenced HEC-RAS model in the manner as described in **Section 9.1**. Geometry and cross section data improvements were made to the HEC-RAS model geometry, improved bridge section representation, corrected downstream reach lengths and levee walls added for Barron and Adobe Creeks.

9.4 STEVENS CREEK.

Stevens Creek flows directly into the San Francisco Bay through the City of Mountain View, with a drainage area of 30 square miles. The study reach consists of an earthen trapezoidal channel throughout, with levees and floodwalls downstream of Hwy 101 to the San Francisco Bay. The creek borders Moffett Field downstream of Hwy 101 to the east and Shoreline Park to the west.

The existing SCVWD HEC-RAS model for Stevens Creek was supplemented with a 2007 District survey of the reach downstream of Highway 101 to the San Francisco Bay. The District contracted Bestor Engineers, Inc. and Sea Surveyor to survey Stevens Creek from Hwy 101 to the bay. The survey included topographic and bathymetric data and was completed in July 2007.

The data was used to create the Stevens Creek HEC-RAS model by combining with the SCVWD model upstream of Hwy 101. The upstream portion of the existing SCVWD hydraulic model was georeferenced in the same manner as described in **Section 9.1**. Additionally, geometry and cross section data improvements were made to the HEC-RAS model geometry, improved bridge section representation, missing bridges added, corrected downstream reach lengths and the fish ladder under Hwy 85 were modified.

9.5 PERMANENTE CREEK.

Permanente Creek flows directly into the San Francisco Bay, with a drainage area of 17 square miles, through a highly urbanized City of Mountain View. The study reach consists of U-frame & trapezoidal concrete channel, in the urban areas and an earthen trapezoid channel through the Bay lands/salt ponds.

A large portion of Permanente Creek was georeferenced in the same manner as the District reach of Coyote Creek. For the reach between Middlefield Road and the bay, the 1998 SCVWD x, y, z survey points were available in a text file format. These points were converted to a shapefile and loaded into HEC-RAS. This allowed the cross sections to be georeferenced with some accuracy. The remaining reach from Middlefield Road to Villa Street, was georeferenced in the manner as described in **Section 9.1.**

Additionally, geometry and cross section data improvements were made to the HEC-RAS model geometry, missing bridges added and corrected downstream reach lengths. Additionally, twelve culverts between Villa Street and Hwy 101 were converted from bridges to culverts to more accurately account for the losses through the culvert.

9.6 SUNNYVALE COMPLEX, GUADALUPE SLOUGH AND TRIBUTARIES.

The SCVWD refers to this drainage basin as the West Valley Watershed. The West Valley Watershed includes Sunnyvale East, Sunnyvale West, Calabazas Creek and San Tomas Aquino, which are all tributaries to Guadalupe Slough, for a combined watershed of 77 square miles. San Tomas Aquino and Calabazas Creek account for approximately 88% of the watershed, originating in the lower ranges of the Santa Cruz Mountains.

Due to the connectivity of all four creeks flowing to Guadalupe Slough before entering the San Francisco Bay the individual SCVWD models were combined into one georeferenced HEC-RAS model.

9.7 SAN TOMAS AQUINO AND GUADALUPE SLOUGH.

San Tomas Aquino is the largest tributary to the Guadalupe Slough within the West Valley Watershed, encompassing 44.9 square miles. The watershed originates in the lower range of the Santa Cruz Mountains, flowing in a northerly direction, through the Cities of San Jose and Santa Clara before it reaches the Guadalupe Slough. The

Guadalupe Slough flows through the former Alviso Salt Pond Complex before reaching the San Francisco Bay. Most of the watershed is located on the valley floor and is highly urbanized. The San Tomas Aquino study reach is located between the confluence with Guadalupe Slough and Highway 101. The study reach consists of a leveed earthen trapezoid channel.

The District contracted Bestor Engineers, Inc. and Sea Surveyor to survey San Tomas Aquino Creek from Hwy 101 and the east/west portion of Guadalupe Slough to it's confluence with Sunnyvale East. The survey included topographic and bathymetric data and was completed in July 2007. The data was used to create the San Tomas Aquino and the east/west portion of Guadalupe Slough HEC-RAS Model. This data was combined with the Guadalupe Slough and tributaries or Sunnyvale Complex HEC-RAS model to create a single model of the West Valley Watershed system within the study area.

9.7.1 CALABAZAS CREEK.

Calabazas Creek flows through the Cities of Santa Clara and Sunnyvale to the Guadalupe Slough and out to the San Francisco Bay just downstream of San Tomas Aquino Creek. The Calabazas watershed contributes 21 square miles, approximately 27% of the West Valley Watershed. Calabazas Creek is a highly urbanized creek consisting of floodwalls, levees and a combination of earthen trapezoidal and concrete channel in the study area.

Additional geometry and cross section data improvements were made to the HEC-RAS model geometry, improved bridge section representation, corrected downstream reach lengths, corrected river station names.

9.7.2 SUNNYVALE EAST AND SUNNYVALE WEST/MOFFATT CHANNELS

The Sunnyvale East and West Channels originate in the City of Sunnyvale and all flow is conveyed via storm drains to the channels. These channels were built in the 1960's by the SCVWD to convey flood flows from the City of Sunnyvale storm drain system to the San Francisco Bay via the Guadalupe Slough. The Sunnyvale East and West watershed is fully urbanized and consist of an open channel trapezoid channel.

Sunnyvale West flows into the Moffatt Channel and then to the Guadalupe Slough and then out to the San Francisco Bay with a contributing watershed of 2.8 square miles. Sunnyvale East flows directly into the Guadalupe Slough downstream of Calabazas Creek. Sunnyvale East has a drainage area of 7.25 square miles.

Both the Sunnyvale East and Sunnyvale West Channels HEC-RAS models were developed by the SCVWD from surveys conducted in August 2006. The model geometry was created using the HEC-GeoRAS extension in ArcMap which creates a spatially represented stream alignment and geometry. The data was then directly imported into HEC-RAS.

All input parameters, horizontal coordinate system, vertical datum, bridge/culvert dimensions, in-stream structures, reach lengths, Manning's n-values and expansion and contraction coefficients were input by the SCVWD.

The District only made modifications to the Sunnyvale West Channel in the Moffatt Channel, just upstream of the confluence with Guadalupe Slough. The original cross sections did not extend from top-of-levee to top-of-levee and were originally cut perpendicular to the low flow channel, not the full flow channel. These cross sections were deleted and new cross sections were re-cut in GIS using GeoRAS by the District using the SCVWD survey data.

9.8 GUADALUPE RIVER, ALVISO SLOUGH.

The Guadalupe River originates in the Santa Cruz Mountains and flows directly into the San Francisco Bay, via the Alviso Slough. The Guadalupe River basin is characterized by steep slopes in the mountains with a large, wide valley. The valley area is relatively flat and highly urbanized. The river flows though the heart of Silicon Valley and downtown San Jose. The drainage basin is approximately 160 square miles and 144 square miles at the confluence with Los Gatos Creek. . Major tributaries to the Guadalupe River include the Los Gatos Creek, Canoas Creek, Ross Creek and Alamitos Creek watersheds. The Guadalupe River Basin is the second largest watershed in the study area.

The Corps of Engineers downtown Guadalupe River project study area includes approximately 2.5 miles of channel improvements and recreation trail for the reach of the Guadalupe River between Hwy 880 adjacent to downtown, San Jose. This project was designed to prevent flooding for a 1% chance exceedance flood event.

The SCVWD Guadalupe River and Alviso Slough HEC-RAS model was based on 1996 survey data. Since then the SCVWD had performed surveys in 2004 and 2005 of the Alviso Slough area. This data was obtained by the district and used to update the HEC-RAS model. The Guadalupe River model upstream of Gold Street was georeferenced in the same manner as described in **Section 9.1**.

Additionally, geometry and cross section data improvements were made to the HEC-RAS model geometry, improved bridge section representation, corrected pier dimensions, missing bridges added, corrected downstream reach lengths, added lateral weir to Pond A8 and adjusted floodwall heights.

9.9 COYOTE CREEK.

Coyote Creek originates in the Diablo Mountain Range and flows in a northeasterly direction though the cities of Morgan Hill, San Jose, and Milpitas before flowing into the San Francisco Bay. Coyote Creek is bounded by the Guadalupe River Watershed on the west and by the Diablo Mountain Range on the East. The Coyote Creek watershed is the largest watershed within in the study area, 308 square miles. The Coyote Creek study area is part of a Corps of Engineers, Sacramento District Project. The Corps designed and built the study reach upstream of Hwy 237 and the SCVWD designed and built the reach downstream of Hwy 237.

This project was designed to prevent flooding for a 1% chance exceedance flood event. The project consists of bypass channel with levees, alternate side overflow channels with offset levees and crossovers. During low flows the flows move along the natural channel to the bay. However during high flow events, the Coyote Creek bypass moves flood waters to the bay, bypassing the natural channel, just upstream of Lower Penitencia Creek.

CAD data was obtained from the Sacramento District. The cross section cut lines in the CAD files were converted to a shapefile and loaded into HEC-RAS. This allowed the cross sections upstream of Hwy 237 to be georeferenced with some accuracy in HEC-RAS. Downstream of Hwy 237 no data was available and there are no bridges for reference for the entire reach. Therefore, cross sections georeferencing was based on downstream cross section distances, by features that could be determined from the aerial photos, such as levees and the low flow channel, using the General Design Memorandum (GDM) and site visits.

Additional geometry and cross section data improvements were made to the HEC-RAS model geometry, improved bridge section representation, corrected downstream reach lengths, corrected river station names and revised cross sections to include overbank regions.

Two major changes to the models obtained from the SCVWD, was the relocation of Tasman Drive and modifications to the Hwy 237 bridge. The SCVWD updated the original design HEC-2 models, which included the addition of Tasman Drive and a new Hwy 237 bridge. The Hwy 237 bridges were modified based on site visits and the GDM. It was determined from the GDM and aerial photos that Tasman Drive was placed in the wrong location in the 2003 model. This was resolved by moving the bridge to its correct location approximately 1400-feet downstream.

9.10 LOWER PENITENCIA CREEK.

Lower Penitencia Creek is located in northeast San Jose. Lower Penitencia Creek is an urban creek, originating in the cities of Milpitas and San Jose. It is boarded by the Coyote Creek watershed to the west and the Berryessa watershed to the east. The creek consists of earth and concrete trapezoid channel. The watershed is approximately 24 square miles. Lower Penitencia Creek flows into Coyote Creek downstream of the Coyote Creek bypass, through the City of Milpitas.

Additional geometry and cross section data improvements were made to the HEC-RAS model geometry, improved bridge section representation, including updating the Hwy 880 bridge crossing and corrected downstream reach lengths.

9.11 FREMONT FLOOD CONTROL CHANNEL AND SCOTT CREEK

The Fremont Flood Control Channel is located within the City of Fremont, Alameda County. The Fremont Flood Control Channel is a small urban channel with a drainage area of 1.57 square miles. The Fremont Flood Control Channel flows from the eastern foothills of into Coyote Creek, just downstream of Lower Penitencia Creek, before flowing to the San Francisco Bay. The channel is located within the ACFCWCD designated area Zone 6 and is referred to as Zone 6, Line B.

The HEC-2 model obtained from ACFCWCD only contained cross sections, based on an ACFCWCD 2003 survey. The cross sections and bridges/culverts were checked against the CAD cross section drawings for accuracy and corrected to match the CAD cross section drawings as necessary. Bank stations were fixed or added in each of the cross sections, and the Line B stream alignment was imported into the model from the CAD file. Channel downstream reach lengths and stationing for each cross section were revised to match the stationing shown in the CAD cross section file.

Scott Creek (Line A) collects runoff from a watershed that is east of I-880 and stretches into the hills east of I-680. The manmade channel begins east of I-680, and becomes a culvert for about a mile before flowing into a steep trapezoidal channel downstream of Warm Springs Rd. The channel makes a sharp bend to the right at the railroad tracks and future BART crossing. The channel widens and deepens just downstream of the BART crossing. It discharges into a storage area immediately upstream of I-880. The runoff then flows through a quadruple 12' x 3' box culvert under I-880 which is prone to siltation. The invert of the culvert is approximately 1.5 feet below the bottom of the creek downstream of I-880. The current condition of Scott Creek upstream of I-880 is severely overgrown, and as a consequence, conveyance in this reach is highly limited. The area downstream of I-880 is a leveed tidal marsh and stormwater detention area, a portion of which is slated for development as Phase II of the Bayside Business Park.

Scott Creek then flows into the Fremont FCC through a double 36" flap-gated CMP. In addition to stormwater runoff in the channel, water levels in Fremont FCC are controlled by tide, storm surge and flow in Coyote Creek. Flows of approximately 1500 cfs in Coyote Creek, which are typical during large events, will create a condition where there will be no gravity drainage until the water surface in the Scott Creek storage area reaches an elevation of 8.75 feet NAVD 88. Therefore, even when a large precipitation event is not coincident with an extreme high tide and storm surge, the majority of the flow in Scott Creek will be detained in this area and slowly drain out over a period of several days after the high flows in Fremont FCC and Coyote Creek have receded.

The system was modeled using HEC-2 by Schaaf and Wheeler in 1989 prior to the installation of two additional 12' x 3' box culverts under I-880, and again in 2008 using HEC-RAS to design a flow diversion structure to direct flows at the Bayside Business Park into a detention area. The 2008 model extended only to the storage area just upstream of I-880. Earth Tech and HNTB modeled the hydraulics of Scott Creek in the vicinity of the future BART crossing for the Valley Transportation

Authority to design a replacement culvert for the existing wooden trellis bridge in 2009.

The USACE HEC-RAS model of Scott Creek includes portions of both the Schaaf & Wheeler and Earth Tech models. The volumes of the storage areas above and below I-880 were measured using LiDAR and a GIS and these were added to the model. Lateral structures were added to quantify overbank flows along the reach from I-880 to upstream of the BART crossing. The dense vegetation in the channel upstream of I-880 was modeled by increasing Manning's roughness values to 0.1. The I-880 culverts were half filled with sediment when inspected by USACE in 2009 and when inspected for the 1989 Schaaf & Wheeler study. It is expected that sediment deposited in the box culvert will be mobilized during a significant event.

9.12 LAGUNA CREEK AND AGUA CALIENTE CREEK

Laguna Creek and Agua Caliente Creeks are located within the City of Fremont, Alameda County. Laguna Creek flows from Lake Elizabeth in Fremont Central Park into Mud Slough before flowing to the San Francisco Bay with a drainage area of 24.6 square miles. Agua Caliente is a tributary to Laguna Creek with a tributary drainage area of 2.63 square miles. Agua Caliente consists of a straightened trapezoid channel.

Agua Caliente Creek enters Laguna Creek just before Laguna Creek makes a ninety degree turn towards Mud Slough. Both creeks are also located within the ACFCWCD designated area Zone 6. Laguna Creek is also referred to as Zone 6, Line E and Agua Caliente is also referred to as Zone 6, Line F by the ACFCWCD.

The Laguna Creek HEC-RAS model reviewed and some modifications were found to be necessary. The original model is based on surveys completed before the *Laguna Creek (Zone6, LineE) Flood Control Project Alternatives Analysis and Feasibility Study, URS April 2003*, report and it was necessary to make modifications to reflect changes along the channel since then. Discrepancies in culvert and bridge lengths at several channel crossings were inconsistent with aerial photos. The widths and the distances to upstream cross sections for these bridges and culverts were corrected to what was measured on background aerial images in HEC-RAS.

Modifications to the HEC-RAS model for Laguna Creek include widening the Cushing Parkway bridge and culvert (widened or rebuilt sometime after April 2003) and replacing the downstream cross section for this bridge by using a copy of the upstream cross section. Added ineffective flow lines needed to be added at cross sections surrounding culverts to restrict their active flow areas at these contracting or expanding flow points.

The Agua Caliente Creek, Line F, tributary was based on a HEC-2 model obtained from the ACFCWCD. First, ACFCWCD HEC-2 files of Line F West and Line F East were imported into HEC-RAS, and the quality of the models was checked. The cross sections in the models were based on a 2003 ACFCWCD survey. The District obtained the 2003 survey CAD files from the ACFCWCD and compared the model cross sections with the AutoCAD cross sections and were found to be nearly accurate except for a small number of points. These points were corrected to match the CAD depictions.

Downstream reach lengths for each cross section were adjusted to match the stationing shown in the CAD cross section files, and then the cross sections were georeferenced along the imported CAD stream alignments. Because there are no significant bends or curves in the Line F West and Line F East models, downstream reach lengths at each cross section were assigned the same value for the channel, the left overbank, and the right overbank. Bridges and culverts were then added with dimensions estimated based on ACFCWCD's CAD cross section drawings and the measurements from the Districts May 2008 site visit. Manning's *n* roughness values and contraction/expansion coefficients were set for the cross sections and culverts along the channel. The model was imported to the Laguna Creek model to create one HEC-RAS model of the creek system.

9.13 EXISTING HYDRAULIC MODEL DATA SOURCES

Existing data relating to the study area were reviewed for developing topographic model inputs or verifying the existing models. The data sources used in updating the existing SCVWD and ACFCWCD HEC-2/HEC-RAS models are as follows.

- *2005 USGS South Bay bathymetric survey*. These data cover most of the deep water areas south of Coyote Point. Data was collected in a joint effort with NOAA-COOPS, Sea Surveyor, Inc. and Tucker & Associates.

- *1998 SCVWD Permanente Creek cross section survey (Choy, 2007)*. Cross-sections do not include bathymetry in the tidal portion of the channel. (File: SCVWDPermSTPl3ft_xyNAVD.txt)

- *SCVWD Matadero Creek as-built CAD files (Mark Thomas & Company, 2005)*. Extensive set of CAD files containing 11 April 2003 design plans with as-built edits dated through 16 September 2005. As-builts differed substantially from the original design plans. Located downstream from highway 101, the bypass redirects high flows to a discharge point just north of the municipal work yard.

- *1999 Towill topographic survey of Palo Alto Flood Basin (Towill, 1999)*. Photogrammetric survey conducted in 1999 for MacKay and Somps.Work done in conjunction with Schaaf & Wheeler study for SCVWD (Schaaf & Wheeler, 2002). Ground control provided by MacKay and Somps..

- *Lower Penitencia Creek at California Circle and I-880 (SCVWD Transmittal, Chung, 2007).* Improvement Plans, Dixon Landing Business Park, Sheet 3 of 14. (1983.) Lower Penitencia Creek Hwy 17 Plan and Profile, Sheet 4 of

43. (1984). State of California Dept. of Transportation Project Plans for Construction of State Hwy 880. (7pp) (2000). All drawings received in hard copy.

- *1997 Caltrans Coyote Creek Improvements at Hwy 237. (Chung, 2007)* Pages C-3, P-2, P-29, P-30. Drawing received in hard copy.

- *1995 USACE Sacramento District Coyote Creek CAD files.(CESPK, Twiss)* CAD files contain design cross sections and with project topography from Hwy 237 to Montague Expressway.

- *SCVWD Sunnyvale East and Sunnyvale West Survey. July 2006.* CAD files and x, y, z data point files received.

- *SCVWD cross-sections of Lower Guadalupe River project.* Post construction cross-section drawings dated late 2004 and early 2005 (SCVWD, 2004, 2005). Survey conducted by SCVWD survey section. All drawings are hard copy and were used as reference for flood wall crest elevations.

- *2005 SCVWD contour map of the Lower Guadalupe River*. A contour map (Figure 16) and profile of the Lower Guadalupe River Flood Control Project downstream of highway 237 (Dyer, 2006).

- *2003 ACFCWCD Aqua Caliente Cross Sections and Stream Alignment.* CAD file containing the stream alignments and cross sections of Lines F West, F East, and B.

- *Fremont Flood Control Channel As-Builts.*

- o 2004 SCVWD topography survey of Alviso Slough (Chung, 2006). Cross-sections surveyed by aerial mapping methods, every 60 meters along length of Alviso Slough.

- o 2007 Bestor / Sea Surveyor San Tomas Aquino Survey (Hink, 2007). A new bathymetric and topographic survey was conducted of Stevens Creek, San Tomas Aquino Creek and the east-west portion of Guadalupe Slough. Work was conducted for the USACE, San Francisco District.

- o *Scott Creek Bypass Channel Drainage Report (Schaaf & Wheeler, 2008).* A proposed business park will modify the existing drainage of Scott Creek downstream of I-880. This report documents the most likely future design.

- o *Line A (Scott Creek) 65% Design SVRT Line Segment Project.* Modification of the existing railroad crossing was underway at the time of this study. Design specifications were used in hydraulic modeling.

10.0 HYDRAULIC MODEL CALIBRATION/VERIFICATION AND SENSITIVITY ANALYSIS.

HEC-RAS uses a number of user-defined input parameters to perform steady flow computations, most notably the Manning's n-value. The recommended value ranges are available, but selection of the final value is dependent on the specific application and the modeler's judgment. The validity of the selected values is usually checked by comparing the model results with measured flow data. If necessary, the model parameters are adjusted to obtain the best agreement between the modeled and measured data. This process of adjusting model input parameters is called "model calibration". However, there are several creeks where no flow and stage data are available for calibration. Sensitivity analyses were therefore performed on a range of modeling parameters to evaluate the reasonableness of the model results. The parameters tested included: (1) Manning's roughness coefficient, and (2) flow. Sensitivity analyses consisted of changing selected modeling parameters (while keeping other modeling parameters unchanged) and assessing the change in the simulated results.

Various high water marks, gage data and observed data were available for calibration and verification for six creeks, Calabazas Creek, Coyote Creek, Guadalupe River, Matadero Creek, Stevens Creek and Sunnyvale East, in Santa Clara County. For each of these streams, the steady flow hydraulic model was both calibrated by adjusting the Manning's n-value to attain reasonable agreement between the measured data and the simulated water surface profile. Comparison of the calibrated water surface elevations between the simulated results and the observed data are summarized in **Table 11.** The results are also shown in **Plates 19-24** for the calibrated models.

Model verification was completed for each of the calibrated creeks. In general, the predicted water surface elevations show good agreement with the observed data for all of the model runs. Additionally, it was shown that the downstream boundary condition has a limited affect on the water surface profiles upstream of the tidally influenced areas. This is due to a number of channel conditions, drop structures, changes in channel slope and in stream structures such as bridges and culverts that mute the effects of the Bay as you move upstream. The results of the model verification are shown in **Table 12**.

The effects of extreme tidal events on the water surface profiles were examined in the production run portion of the study.

Table 11. Calibration Results, Simulated V. Observed Water Surface Elevations

Creek	Location	Date	Flow (cfs)	Water Surface Elevation (NAVD88, feet)	
				Observed	Simulated
Calabazas	Monroe Ave	2/17/2004	219	48.3	46.7
Calabazas	Monroe Ave	2/14/1986	485	49.3	47.6
Calabazas	Monroe Ave	1/4/2008	1267	51.1	49.6
Calabazas	Monroe Ave	2/25/2004	1900	52.2	50.6
Calabazas	Monroe Ave	1/4/2008	1926	52.2	50.6
Calabazas	Monroe Ave	1/24/1983	2410	52.9	53.6

Calabazas	Monroe Ave	2/17/2004	219	48.4	46.7
Coyote Ck	U/S Hwy 237	2/18/2004	432	11.2	12.5
Coyote Ck	U/S Hwy 237	4/5/2006	1,140	14.5	15.8
Coyote Ck	U/S Hwy 237	1/4/2008	1460	15.7	16.2
Coyote Ck	U/S Hwy 237	1/24/2000	2550	15.8	17.2
Coyote Ck	Btw Montague Expwy and Charcot Ave	1/27/1997	8,000	37.9	38.8
Coyote Ck	D/S Charcot Ave	1/27/1997	8,000	41.1	41.5
Guadalupe	Hwy 101	2/2/2004	623	23.82	24
Guadalupe	Hwy 101	2/18/2004	1037	24.82	24.9
Guadalupe	Hwy 101	12/31/2005	1,760	25.91	26.1
Guadalupe	Hwy 101	2/25/2004	2911	27.81	27.7
Guadalupe	Hwy 237	1/26/1997	5,470	13.1	13.3
Guadalupe	Hwy 237	1/9/1995	9,290	15.4	15.4
Guadalupe	Tasman Dr	1/26/1997	5,470	16.4	15.8
Guadalupe	Tasman Dr	1/9/1995	9,290	18.4	19
Guadalupe	Montague Expwy	1/26/1997	5,470	19.0	19.2
Matadero Ck	El Camino Real	1/31/1996	153	23.88	24.1
Matadero Ck	El Camino Real	1/4/2008	454	24.96	25.4
Matadero Ck	El Camino Real	2/2/1998	2560	29.76	31.3
Stevens Ck	Central Ave.	2/25/2004	308	59.82	59.90
Stevens Ck	pedestrian bridge just u/s of gage station	1/4/2008	394	59.08	59.80
Stevens Ck	Central Ave.	2/25/2004	664	58.83	60.70
Stevens Ck	Central Ave.	1/1/2004	839	59.40	61.20
Sunnyvale East	Hwy 101	2/17/2004	39	19.53	19.3
Sunnyvale East	Hwy 101	1/24/1983	77	21.09	19.7
Sunnyvale East	Hwy 101	2/26/2004	140	20.69	20.3
Sunnyvale East	Hwy 101	2/2/2004	340	21.65	22.3
Sunnyvale East	Hwy 101	2/25/2004	527	22.63	24.2

Table 12. Verification Results, Simulated V. Observed Water Surface Elevations

Creek	Location	Date	Flow (cfs)	Water Surface Elevation (NAVD88, feet)	
				Observed	Simulated
Calabazas	Monroe Ave	1/24/1983	2760	53.4	54.3
Calabazas	Monroe Ave	2/14/1986	3280	54.0	55.3
Calabazas	Monroe Ave	2/26/2004	318	48.7	47.1
Calabazas	Monroe Ave	1/4/2008	1218	50.4	49.5
Coyote Ck	U/S Hwy 237	12/16/2002	1,460	15.7	16.2
Coyote Ck	U/S Hwy 237	2/25/2004	691	12.7	13.7
Guadalupe	Hwy 101	3/25/2004	506	23.4	23.6
Guadalupe	Hwy 101	1/4/2008	5200	29.7	29.8
Matadero Ck	El Camino Real	1/13/1993	280	24.3	24.1
Matadero Ck	El Camino Real	1/25/2008	650	25.6	26.1
Stevens Ck	Central Ave.	2/25/2004	359	60.2	60.1
Stevens Ck	Central Ave.	1/1/2004	288	59.8	59.8
Sunnyvale East	Hwy 101	1/24/1983	541	24.4	24.3

As part of this study, on January 4, 2008 the SCVWD collected stream flow, velocity and depth data for calibration purposes. The January 4, 2008 storm event was fairly mild and no flooding occurred. The additional data was used in the calibration/verification analysis for Sunnyvale East, Calabazas Creek and Stevens Creek. However, the data collected for Permanente Creek, Adobe Creek and Barron Creek was insufficient to complete the calibration/verification of the HEC-RAS model, as only one data point was collected. For these creeks the models were calibrated to the single data point, in addition to the sensitivity analysis. The results of the model calibrations to the January 4, 2008 event are shown in **Table 13**.

Table 13. January 2008 Event - Simulated V. Observed Water Surface Elevations

Creek	Location	Date	Flow (cfs)	Water Surface Elevation (NAVD88, feet)	
				Observed	Simulated
Adobe Ck	Upstream Middlefield	1/4/2008	296	9.2	9.5
Barron Ck	Upstream Middlefield	1/4/2008	157	11.8	12.1
Permanente	Upstream Middlefield	1/4/2008	628	27.99	28.34

Sensitivity analyses were performed for the remaining creeks, Adobe Creek, Barron Creek, Permanente Creek, Sunnyvale West, San Tomas Aquino, Lower Penitencia, Fremont Flood Control Channel, Laguna Creek and Agua Caliente. These creeks are un-gaged and do not have high water mark data available.

The sensitivity analysis approach included determining sensitivity of the upper and lower limits to Manning's n-value on the water surface profile. The Manning's n-values were varied between published limits for the appropriate channel surface type. Additionally, the flow rates for the 50-, 20-, 10- , 4-, 2- and 1- percent events were varied by 10% and 15% above and below the median flows to determine the relationship with the water surface elevation. The downstream boundary condition was set to MHHW NAVD88 so that the stability of the water surface profiles could be determined with respect to the roughness coefficient.

In general, for each creek the variation of flow has a less significant effect on the water surface elevation profile causing a minor shift of the profile, on the order of 0.5-ft. Changes to Manning's n-value have the most significant impacts to the water surface elevation. Overall, variation in the Manning's n-value results in a change in water surface profile on the order of 1-ft.

In summary, the HEC-RAS models are producing acceptable results. However, it would be beneficial to continue with a data collection effort to complete calibration/verification of the Adobe Creek, Barron Creek, Permanente Creek, Sunnyvale West, San Tomas Aquino, Lower Penitencia, Fremont Flood Control Channel, Laguna Creek and Agua Caliente HEC-RAS models. Additional, high flow data would improve the Calabazas Creek, Coyote Creek, Guadalupe River, Matadero Creek, Stevens Creek and Sunnyvale East HEC-RAS models for larger flow events.

A detailed summary of the calibration and verification data used in this study are included in **Appendix B**. For a detailed discussion on this effort, refer to *South San Francisco Bay Shoreline Study (SSFBSS) Hydraulics and Hydrology Support Final Submittal, Moffat & Nichol, September 2008*.

11.0 SEA-LEVEL CHANGE

Coastal zones are vulnerable to climate variability and change and the San Francisco Bay is no different. Rising sea levels inundate wetlands and other low-lying lands. It can also increase salinity in groundwater tables and the chance of flooding in coastal areas. Historically, in the past 150 years the San Francisco Bay has risen 7 inches. To incorporate future sea-level change, the rate of change evaluated for this report is based on NRC #1 (0.72 feet from year 0 to year 50).

12.0 COINCIDENT FREQUENCY ANALYSIS

Flooding is often the result of multiple factors working together. Usually the probability of a flood event itself will be unknown, even if the contributing factors have known probability distributions. Any situation that requires water levels to be predicted as a function of several variables: pre-storm reservoir stage and storm inflows, river stage influenced by backwater or tides, wintery combinations of snow melt plus ice jams and/or storm runoff, and tidal phase before a storm surge.

In some cases it is clear that several conditions will contribute to a flood stage but their statistical relationship may not be apparent. These simultaneous or "coincident" causes may be perfectly correlated, that is statistically more likely to occur together, or they may be independent. However, because the variable of interest, flood level, is a function of each of the contributing causes, their joint probability must be used to predict flooding frequency. Therefore, the purpose of the coincident frequency analysis for this study is to calculate the joint probability of the peak tide and peak stream discharge effects which coincide at a particular location of interest along the stream.

12.1 CORRELATION OF TIDES AND STREAM DISCHARGE ANALYSIS

To apply the Law of Total Probability in the form as discussed above, it is necessary to determine if the variables of tide and stream discharge are dependent, independent or somewhere in-between. To determine the coincidence of the peak tide and peak discharge a correlation analysis was performed.

Three stream gages within Santa Clara County were used for the correlation analyses. The stream gages used for this analysis are located on San Tomas Aquino Creek, Guadalupe River and San Francisquito Creek, shown on **Plate 25**. Descriptions of the stream gages used for the analyses are included in Table 14. These gages were chosen for the amount and quality of data and geographic location. The stream gage on San Tomas Aquino Creek has been in continuous operation since 1955, with only minor data gaps. Fifty one years of electronic data was readily available for the analysis on San Tomas Aquino Creek. The stream gages on the Guadalupe River and San Francisquito Creek have been in operation since the 1930’s, but the data prior to 1988 is not readily available or in electronic format and very difficult to obtain. However, since 1982, 1983 and 1986 were years of large storm events, this data was obtained from the National Archives by the USGS.

Table 14. Correlation Stream Gages

Gage Name	Gage # /Operator	Correlation Years	Number of Years
San Tomas Aquino at Williams Rd	#24 / SCVWD	1955 - 2006	51
Guadalupe River at San Jose	11169000/ USGS	1982, 1983, 1986, 1988- 2003	21
Guadalupe River above Hwy 101 at San Jose	11169025/USGS	2003 - 2006	
San Francisquito Creek at Stanford University	11164500/USGS	1982, 1983, 1986, 1988-2006 (Missing 1998)	20

12.2 SAN FRANCISCO BAY WATER LEVELS

The stage of the San Francisco Bay at the Dumbarton Bridge was developed due to it's proximity to the study area. Water level data at the Dumbarton Bridge is based on hourly data for the San Francisco Bay Presidio (Presidio) tide gage for the period of record, 1900 to 2006. No data gaps were noted from the data. The Presidio Tide gage data was corrected for the Dumbarton Bridge stage. The correction was computed as the following:

$$\text{Dumbarton Bridge (DB)} = (\text{SF-pred} - \text{MSL-SF})*1.46 + \text{SF-res} + \text{MSL-DB}$$

(Equation 12.1)

Where:

DB = Dumbarton Bridge Stage (feet)

SF-pred = SF Bay Presidio tide gage predicted value (feet)

MSL-SF = Mean Sea Level at the SF Bay Presidio tide gage (feet)

SF-res = SF Bay Presidio tide gage residual (feet)

MSL-DB = Mean Sea Level at the Dumbarton Bridge (feet)

The data was also detrended, where the detrending of the data was computed as the following:

$$\text{Correction} = 0.0064 * \text{Year} - 12.84 \quad \text{(Equation 12.2)}$$

Where:

Year = the year of interest. i.e. For the year 1900, Correction = -0.68 feet.

Using HEC-DSSVue, the tide gage data was converted from Greenwich Mean Time, (GMT) to PST, by subtracting eight (–8) hours from GMT. Additionally, to account for the phase lag from the San Francisco Presidio tide gage to the south Bay an hour (+1) was added to the tide gage data.

Hourly data from the Presidio tide gage for the same time period of the both stream gages was also brought into DSS. The hourly tide gage data was then converted to 15 minute data in DSS to allow for the ease in which to obtain the tide stage at the corresponding stream flow peak. The peak stream flow data was used as a pattern in which to pull out the corresponding tide stage for each of the creeks used in the analysis.

12.3 SAN TOMAS AQUINO CREEK CORRELATION ANALYSIS

The San Tomas Aquino Creek stream gage, #24, is located approximately 8 miles upstream of Highway 237, with a drainage area of 13.4 square miles. The gage is located 1500-feet upstream of Williams Road on the right bank. The stream gage captures 30% of the runoff from the watershed. San Tomas Aquino Creek enters the Guadalupe Slough before flowing to the San Francisco Bay approximately 6.8 miles downstream of Hwy 237. At Hwy 237 the creek has a drainage area of 44.9 square miles. The stream gage is operated by the SCVWD.

Stream gage data was obtained from the SCVWD for stream gage, SF#24. The gage has been in continuous operation since October 1955. From 1955 to 1970 the gage collected data every 5 minutes and from 1971 to present data has been collected in 15 minute intervals. The entire data set from 1995 to 2007 was available for the analysis. The data was reviewed for data gaps, reasonableness and assumed to be in Pacific Standard Time (PST). Several data gaps were identified and a period of irregular data was found. Missing data was given a value of "0", zero. These data problems are noted in the Table 15.

The 5 minute data was converted to 15 minute data in HEC-DSS, to reduce the number of data points, to approximately 7,000,000, and create a single cohesive data set. HEC-DSSVue has a limited ability to store only 2GB of data in a single DSS file and therefore, several DSS files were created for the 15 minute data stream gage data.

Since the stream gage only captures 30% of the runoff from the watershed it was necessary to transfer the timing of the peak to Guadalupe Slough. The peak time of travel was determined by referencing the HEC-1 model developed by the SCVWD. The timing of peak at the stream gage (CP5) was subtracted from the time it took the peak to travel to Hwy 237 (CP12). Then the time of travel in the upstream reaches were averaged to determine the time of travel to the computation point in the Guadalupe Slough. The total time of travel from the stream gage to the computation point in the Guadalupe Slough was determined to be 1.25 hours. The 1.25 hours was then added to the time of the peak at the stream gage to determine the time of peak within the Guadalupe Slough.

Table 15. San Tomas Aquino Creek Data Gaps

Date	Problem Identified
03 March 1970, 10:35 – 01 October 1972, 24:00	Missing
08 November 1996, 24:00 – 12 November 1996, 10:15	Missing
06 April 1998, 24:00 – 30 September 1998, 23:45	Missing
10 January 2005, 24:00 – 01 February 2005, 23:45	Single value reported for each day.
02 February 2005, 24:00 – 02 February 2005. 8:15	Missing

Once the corresponding peak and stage values were determined the data was sorted and values below 1000 cfs were thrown out. Flows below 1000 cfs were not considered significant enough to include in the analysis, below the 1.5-yr event at the stream gage site.

There are a total of 95 values on San Tomas Aquino Creek that are greater than 1000-cfs. The ten largest flood events on San Tomas Aquino Creek and the corresponding tidal stage are shown in Table 16.

Table 16. San Tomas Aquino 10 Largest Flood Events & Corresponding Tide Levels Since 1955.

Date/Time	San Tomas Aquino		SF Bay
	Flow (cfs)	Return Period (years)	Stage (feet, NAVD88)
14Jan1978 1930	2990.00	7%	3.01
09Jan1995 0530	2890.00	-	7.52
03Feb1998 0215	2870.00	-	6.72
07Feb1998 1515	2770.00	-	1.05
26Jan1983 2045	2730.00	-	4.21
14Feb1986 1515	2650.00	-	5.75
24Jan1983 0115	2550.00	10%	4.23
19Feb1980 0745	2410.00	-	1.98
24Jan1983 0215	2410.00	-	4.41
07Mar1975 1100	2390.00	7	5.01

These 95 values were plotted versus the corresponding tide stage at the mouth of the stream to determine the correlation between the water levels in the SF Bay at the Guadalupe Slough and the peak flows on San Tomas Aquino creek. The results of the correlation plot indicate that the two variables are independent. It can be seen in **Plate 26**, that the two variables have virtually no correlation, R=0, as such they are considered independent.

Further analysis was conducted to determine the correlation between the tidal surge and the riverine peaks. As shown on **Plate 27**, the results of the correlation analysis between the tidal surge and the San Tomas Aquino Peak flow indicate that the surge and riverine peak have some correlation, but it significantly less than one, R = 0.4 and small enough to ignore and maintain the assumption of independence.

12.4 GUADALUPE RIVER CORRELATION ANALYSIS

The Guadalupe River correlation analysis uses two stream gages, USGS 11169025 and 11169000, with drainage areas of 160 square miles and 146 square miles, respectively. Stream gage 11169000, was located downstream of the confluence with Los Gatos Creek, was relocated stream gage 11169025, to upstream of Hwy 101 in 2003. The current gage is located 300-feet upstream of Highway 101. The Guadalupe River enters the Alviso Slough before flowing to the San Francisco Bay approximately 4.6 miles downstream of Hwy 101.

Stream gage data was obtained from the USGS. The gage has been in continuous operation since October 1930. However, only data from 1982, 1983, 1986 and 1988 to 2007 was available. The data was reviewed for data gaps, reasonableness. The USGS data also takes into account Daylight Savings Time. This was corrected to Pacific Standard Time (PST).

The stream gages capture 95% and 82% of the runoff from the watershed, respectively. Again, it was necessary to transfer the timing of the peak to Alviso Slough. The peak time of travel was determined by referencing a preliminary Hydrology model of the Guadalupe River under development by the District. The time of travel in the upstream reaches were averaged to determine the time of travel to the computation point in the Guadalupe Slough. The total time of travel from the stream gage to the computation point in the Guadalupe Slough was determined to be 2.75 hours and 3 hours, respectively. The time of travel was then added to the time of the peak at the stream gage to determine the time of peak within the Guadalupe Slough.

Once the corresponding peak and stage values were determined the data was sorted and values below 2700 cfs were thrown out. Flows below 2700 cfs, the 50% chance exceedance flood event, were not considered significant enough to include in the analysis.

There are a total of 59 values on the Guadalupe River which are greater than 2700-cfs. The ten largest flood events on Guadalupe River and the corresponding tidal stage are shown in the Table 17.

Table 17. Guadalupe River 10 Largest Flood Events & Corresponding Tide Levels Since 1982

Date	Time at Mouth (PST)	Flow (cfs)	USGS Gage	% Chance Exceedance	Stage (NAVD88,feet)
3/10/1995	13:00	10400	11169000	3%	2.76
1/9/1995	23:00	9290	11169000	5%	3.55
2/19/1986	2:30	9140	11169000	5%	6.61
2/19/1986	0:00	7890	11169000	6%	5.31
2/3/1998	2:15	7510	11169000	7%	9.46
3/31/1982	12:30	7340	11169000	7%	2.31
1/24/1983	4:15	7130	11169000	8%	8.64
2/14/1986	15:15	7000	11169000	8%	5.41
1/26/1983	20:45	6970	11169000	9%	7.43
3/9/1995	14:15	6540	11169000	11%	4.18

These 59 values were plotted versus the corresponding tide stage at the mouth of the stream to determine the correlation between the water levels in the SF Bay at the Alviso Slough and the peak flows on Guadalupe River. The results of the correlation plot indicate that the two variables are independent. It can be seen in **Plate 28**, that the two variables have virtually no correlation, R=0, as such they are considered independent.

Further analysis was conducted to determine the correlation between the tidal surge and the riverine peaks. As shown on **Plate 29**, the results of the correlation analysis between the tidal surge and the Guadalupe River peak indicate that the surge and riverine peak have some correlation, but it significantly less than one, R =0.46, and small enough to ignore and maintain the assumption of independence.

12.5 SAN FRANCISQUITO CREEK CORRELATION

The USGS San Francisquito Creek stream gage 11164500 was used for the correlation analysis. The San Francisquito Creek stream gage has a drainage area of 37.4 square miles and is located 1.1 miles downstream from Los Trancos Creek in the Stanford University golf course. The creek flows directly into the San Francisco Bay.

Stream gage data was obtained from the USGS. The gage has been in continuous operation since October 1950. However, only data from 1982, 1983, 1986 and 1988 to 2007 was available for this analysis. The data was reviewed for data gaps, reasonableness. The USGS data also takes into account Daylight Savings Time. This was corrected to Pacific Standard Time (PST).

The stream gage captures 81% of the runoff from the watershed. Again, it was necessary to transfer the timing of the peak the SF Bay. The peak time of travel was determined by referencing the HEC-1 model developed by the SCVWD. The timing of peak at the stream gage (S10) was subtracted from the time it took the peak to travel to downstream computation point at the Palo Alto Airport (S15). The total time of travel from the stream gage to the downstream computation point (S15) was determined to be 0.8 hours. The 0.8 hours was then added to the time of the peak at the stream gage to determine the time of peak near the San Francisco Bay.

Once the corresponding peak and stage values were determined the data was sorted and values below 1200 cfs were thrown out.

There are a total of 38 values on San Francisquito Creek which are greater than 1200-cfs. The ten largest flood events on San Francisquito Creek and the corresponding tidal stage are shown in the Table 18.

Table 18. San Francisquito Creek 10 Largest Flood Events & Corresponding Tide Levels Since 1982

Date	Time at Mouth (PST)	San Francisquito Ck		SF Bay Stage
		Q (cfs)	% Chance Exceedance	
12/31/2005	9:45	4840	8%	8.71
12/16/2002	7:30	3730	14%	7.63
1/26/1983	20:30	3420	17%	3.76
1/24/1983	1:45	3340	9%	4.22
1/9/1995	21:00	3320	9%	5.13
1/2/1997	9:15	3250	10%	5.56
1/1/1997	13:30	3150	11%	3.71
1/13/1993	8:15	3010	22%	3.52
12/10/1996	10:30	2880	25%	8.98
2/7/1999	13:45	2640	33%	2.36

These 38 values were plotted versus the corresponding tide stage at the mouth of the stream to determine the correlation between the water levels in the SF Bay and the peak flows on San Francisquito Creek. The results of the correlation plot indicate that the two variables are independent. It can be seen in **Plate 30** the two variables have virtually no correlation, R=0, as such they are considered independent.

Further analysis was conducted to determine the correlation between the tidal surge and the riverine peaks. As shown on **Plate 31**, the results of the correlation analysis between the tidal surge and the San Francisquito Creek peaks indicate that the surge and riverine peak have some correlation, but it significantly less than one, R = 0.38, and small enough to ignore and maintain the assumption of independence.

12.6 CALCULATIONS AND RESULTS

The coincident frequency analysis predicts the river stage or downstream boundary condition, influenced by tidal stage. This analysis is based on the results of the correlation analysis such that independence between the Bay water levels and peak stream flows exists. The coincident frequency analysis develops a probability for the riverine downstream boundary condition using the method of total probability.

The frequency curve for the downstream boundary condition, the variable of interest, requires a stage-duration curve with a distribution spanning the approximate 0% stage to the approximate 100% probability stage. Hourly tide probability distribution functions at the river mouths were determined in the vicinity of Adobe Creek, Permanente Creek, Stevens Creek, Guadalupe Slough, Alviso Slough and Coyote Creek as part of the Coastal Flooding Uncertainty Analysis for South San Francisco Bay Shoreline Study: without Project Conditions, ERDC. (Letter, in preperation)

The probability distribution functions for the six locations were then discretized into 8 ranges to represent a portion of the range and probability. The probability was divided into ranges and an average of the values in each range was chosen, which then took on the probability of the range. Because of the interest in the higher end of the curve, the curve was discretized in finer detail to more accurately account for the extreme tidal range. The ranges and their associated probabilities values are shown in Tables 19 and 20 for both year 0 and year 50 conditions and also on **Plate 32-38 and 39a-k**, respectively.

The year 50 probability distribution function takes into account sea level rise by adding a constant of 0.72 –feet to each of the probability curves.

Table 19. Probability Distribution Function for Year 0 (feet)

Year 0, Tidal Stage at River Mouth, (NAVD88, Feet)											
Range (probability)	% of Time not Exceeded (Upper limit)	Adobe/ Barron	Coyote	Fremont/ Scott	Guadalupe River	Guadalupe Slough	Laguna	Lower Penitencia	Matadero	Permanente	Stevens
0.2	20.0	-1.10	-1.10	-1.10	-1.04	-0.87	-1.10	-1.10	-0.93	-0.95	-0.98
0.2	40.0	1.68	1.68	1.68	1.71	1.77	1.68	1.68	1.75	1.74	1.73
0.2	60.0	3.42	3.42	3.42	3.42	3.41	3.42	3.42	3.41	3.41	3.42
0.2	80.0	4.83	4.83	4.83	4.81	4.75	4.83	4.83	4.77	4.78	4.79
0.15	95.0	6.26	6.26	6.26	6.22	6.10	6.26	6.26	6.14	6.16	6.18
0.045	99.5	7.73	7.73	7.73	7.67	7.50	7.73	7.73	7.56	7.59	7.62
0.0049	99.99	9.13	9.13	9.13	9.05	8.84	9.13	9.13	8.91	8.95	8.98
0.0001	100.00	10.35	10.35	10.35	10.27	10.05	10.35	10.35	10.12	10.16	10.20

Table 20. Probability Distribution Function for Year 50 (feet)

Year 50, Tidal Stage at River Mouth, (NAVD88, Feet)											
Range (probability)	% of Time not Exceeded (Upper limit)	Adobe/ Barron	Coyote	Fremont/ Scott	Guadalupe River	Guadalupe Slough	Laguna	Lower Penitencia	Matadero	Permanente	Stevens
0.2	20.0	-0.21	-0.38	-0.38	-0.32	-0.15	-0.38	-0.38	-0.21	-0.23	-0.26
0.2	40.0	2.47	2.40	2.40	2.43	2.49	2.40	2.40	2.47	2.46	2.45
0.2	60.0	4.13	4.14	4.14	4.14	4.13	4.14	4.14	4.13	4.13	4.14
0.2	80.0	5.49	5.55	5.55	5.53	5.47	5.55	5.55	5.49	5.50	5.51
0.15	95.0	6.86	6.98	6.98	6.94	6.82	6.98	6.98	6.86	6.88	6.90
0.045	99.5	8.28	8.45	8.45	8.39	8.22	8.45	8.45	8.28	8.31	8.34
0.0049	99.99	9.63	9.85	9.85	9.77	9.56	9.85	9.85	9.63	9.67	9.70
0.0001	100.00	10.84	11.07	11.07	10.99	10.77	11.07	11.07	10.84	10.88	10.92

Current Corps guidance on coincident frequency analysis indicates the goal is to develop a probability distribution for a variable without records, from the observed records of the variables that influence it. The goal of the coincident frequency analysis is to find the single-variate probability distribution of stage at an index point.

The conversion from a multi-variate distribution such to a single-variate distribution requires a weighted summation over a known probability duration curve, the tidal stage at the mouth of the creek in the San Francisco Bay.

To develop a probability distribution of the creek at the chosen index point, the 8 values shown in Tables 19 and 20 for both year 0 and year 50 conditions were input to HEC-RAS as the downstream boundary condition for every exceedance probability to compute the response function of the creek. The index point for each creek is shown on **Plate 40.** The inflow hydrographs and the stage value of the SF Bay tide were used to create a matrix of profile runs to compute the response function for each creek.

The response function is a range of multiple conditional curves, computed in HEC-RAS, which are combined using the Law of Total Probability. The Law of Total Probability is defined as:

$$P(C) = \sum_{B} \left[P(C \mid B) * P(B) \right] \qquad \text{(equation 12.1)}$$

C represents the stage at the index point for each creek. P(C) represents the probability of a particular stage occurring. The goal of the coincident frequency analysis is to calculate P(C).

B must be a contributing variable with a known probability duration curve. In this case, B represents the tidal stage in the San Francisco Bay at the mouth of the creek. P(B) is the probability of a particular B. It would be impossible to run a HEC-RAS model for every value of B, as there could be infinite values, so the stage-probability curve of B must be discretized into a manageable number of B values. The discretization of B must span the range of B and the sections must not overlap. In the case of each of the study creeks, the tidal range was divided into 8 ranges. With the 8 storm frequencies, having 8 tidal ranges resulted in 64 flow-tide combinations, which is a manageable number.

P(C|B) is the probability of a particular C, given that a particular B exists. This can also be called the conditional probability of C given B. So for each B, the probability of C is found and multiplied by the probability of B and these values are summed over the full range of B. This is why variable B requires a full duration probability curve: so that it can be summed across for every value of B.

This computation is repeated multiple times to compute P(C), the probability of stage for the downstream boundary condition, for the range of downstream stage at the index point, variable C. For this study this included C for each standard probability (50-, 20-, 10-, 4-, 2-, 1-, 0.5-, and 0.2-percent chance exceedance). This computation was accomplished by using Microsoft Excel to calculate each C via interpolation. The result is a marginal (single-variate) probability distribution of variable C, which is the effective downstream boundary condition. This is demonstrated on **Plates 41a-k** and **42a-k** which show 3-dimensional views of each of the creek of interest stage as a function of tide and flow for years 0 and 50, respectively.

A detailed description of the coincident frequency analysis methodology can be found in *"Coincident Frequency Analysis for Planning and Design in Urban Areas*", by Faber and Gibson 2005.

The results of the analysis are a single curve of stage versus probability for each river or creek at the defined river station. These curves also represent the downstream boundary condition of riverine influence for the 50-, 20-, 10-, 4-, 2-, 1-, 0.4- and 0.2-percent events. Beyond this point the system is tidally dominated. The results are shown in **Tables 21 and 22** and in **Plates 43a-k and 44a-k**, for years 0 and 50 respectively.

Table 21. Year 0 Coincident Frequency Results – Downstream Boundary Elevation (feet – NAVD88)

Creek	River Station	Location	Percent Chance Exceedance							
			50%	20%	10%	4%	2%	1%	4%	2%
Coyote Creek	73+65	3100-feet downstream diversion structure	9.48	10.64	11.28	11.90	12.58	12.99	13.35	13.57
Guadalupe River	244+81	700-feet downstream SPRR bridge	9.30	11.16	12.26	13.02	13.69	14.16	14.63	14.75
Lower Penitencia	3+76	350-feet upstream confluence with Coyote Creek	7.50	8.61	9.37	10.64	11.52	12.43	13.35	14.00
Permanente Creek	87+93	2000-feet downstream Amphitheatre Pkwy	9.74	11.49	11.96	11.99	12.13	12.15	12.24	12.30
Stevens Creek	48+96	1500-feet downstream Stevens Creek Trail pedestrian bridge	10.58	10.82	10.93	11.04	11.05	11.06	11.06	11.06
Guadalupe Slough	277+74	2700-feet downstream confluence Sunnyvale East	11.85	11.97	12.03	12.07	12.10	12.11	12.13	12.14
Sunnyvale West Channel	106+52	Upstream Bordeaux Drive Bridge	11.15	11.73	12.02	12.26	12.38	12.53	12.68	12.76
Adobe Creek/ Barron Creek	125+65	500-feet downstream Highway 101 Bridge	3.48	5.56	6.40	7.14	7.64	7.89	8.41	8.56
Matadero Creek	99+83	440-feet upstream Frontage Road Bridge	5.88	6.71	7.24	7.84	8.09	8.28	8.59	8.81
Laguna Creek	192+48	80-feet downstream Agua Caliente confluence	7.80	9.02	11.06	11.82	11.99	12.05	12.11	12.15
Fremont Flood Control Channel	24+04	500-feet downstream Highway 880 Bridge	5.36	6.39	7.02	7.66	7.98	8.31	8.66	8.92

Table 22. Year 50 Coincident Frequency Results – Downstream Boundary Elevation (feet – NAVD88)

Creek	River Station	Location	Percent Chance Exceedance							
			50%	20%	10%	4%	2%	1%	4%	2%
Coyote Creek		3100-feet downstream diversion structure	9.48	10.64	11.28	11.90	12.58	12.99	13.35	13.57
Guadalupe River	73+65	700-feet downstream SPRR bridge	9.44	11.21	12.26	13.02	13.69	14.16	14.62	14.75
Lower Penitencia	244+81	350-feet upstream confluence with Coyote Creek	7.50	8.61	9.37	10.64	11.52	12.43	13.35	14.00
Permanente Creek	3+76	2000-feet downstream Amphitheatre Pkwy	9.79	11.49	11.96	11.99	12.14	12.16	12.25	12.30
Stevens Creek	87+93	1500-feet downstream Stevens Creek Trail pedestrian bridge	10.58	10.82	10.93	11.04	11.05	11.06	11.06	11.06
Guadalupe Slough	48+96	2700-feet downstream confluence Sunnyvale East	11.86	11.98	12.04	12.08	12.10	12.12	12.14	12.16
Sunnyvale West Channel	277+74	Upstream Bordeaux Drive Bridge	11.22	11.76	12.05	12.28	12.41	12.55	12.70	12.78
Adobe Creek/Barron Creek	106+52	500-feet downstream Highway 101 Bridge	4.18	6.30	7.09	7.88	8.40	8.67	9.13	9.28
Matadero Creek	125+65	440-feet upstream Frontage Road Bridge	5.97	7.01	7.61	8.20	8.58	8.90	9.26	9.51
Laguna Creek	99+83	80-feet downstream Agua Caliente confluence	7.91	9.25	11.09	11.84	12.00	12.07	12.14	12.18
Fremont Flood Control Channel	192+48	500-feet downstream Highway 880 Bridge	5.52	6.77	7.54	8.15	8.67	9.00	9.37	9.61

From the results of the multiple model runs is can be observed that the creek flow is only driven by the Bay to a point where the creek flow begins to dominate the conditions upstream. Upstream of this point the water surface elevations converge. This usually occurs where there is a change in slope or in-stream structure, otherwise this convergence is gradual. For any storm, how far upstream the creek stage is affected by the tide is a function of how much riverine flow is produced. Essentially, low-flow events are affected by tide levels further upstream than high-flow events. In other words, during larger flow events, the channel flow dominates farther downstream than for low-flow events.

The coincident frequency analysis only applied to the area of the channel where the tide driven water levels and the creek flow meet or commingle. Downstream of the commingling area the water levels are tidally driven and upstream of this area the water levels are dominated by the creek flow. This is demonstrated well in **Plate 45**

The water surface elevations upstream of the commingling area can be determined from the riverine flow for each downstream boundary condition. In other words, the tide has no effect in this area and the probability of the stage in the creek is based on the probability of the storm event with the coincident frequency results acting as the downstream boundary conditions. Conversely, downstream of the index point the stage is dominated by the tides and the probability of the stage is based on the probability of the tides only.

Year 50 sea level change has little affect on the lower exceedance probabilities; there is virtually no change between year 0 and year 50 for the 20 -, 1-, 0.4- and 0.2-percent events between year 0 and year 50. However, it can be seen that sea level rise has the most significant affect on the more frequent storm events. This is shown in Tables 21 and 22.

13.0 HEC-RAS MODEL GEOMETRY ASSUMPTIONS

This study considers the year 0 for the without project conditions to be the year 2017. The existing riverine models represent current conditions, so assumptions were made to account for future projects in the study reaches. The ACFCWCD does not have any projects planned that would affect the channel geometry for the Alameda County, Zone 6 creeks. However, the SCVWD currently have several projects in the planning phase and anticipates completed construction by 2017, year 0.

The only creeks protected from the Bay influence are Matadero, Adobe and Barron Creeks, these three creek flow into the PAFB and floodwaters are stored until tide levels are lower than the water levels in the flood basin. At which time the tide gates are opened to the Bay to release the water from the flood basin. However, for this study a failure was assumed to of occurred along the PAFB levees to determine the affects the Bay water levels would have on the creek, if the failure occurred.

The project features listed in Table 23 were used to modify the hydraulic models to reflect the year 0 and year 50 conditions.

Table 23. Year 0 and Year 50 Riverine Model Assumptions.

Creek	Year 0 Assumptions
Permanente Creek	4-feet high walls, above existing levee crest elevation, between Hwy 101 and Gold Course
Sunnyvale East	7-feet high walls above existing levee crest elevation between Hwy 101 and land ward edge of ponds.
Sunnyvale West	7-feet high walls above existing levee crest elevation between Hwy 101 and land ward edge of ponds.
Palo Alto Flood Basin (PAFB)	Non-functioning flood control facility. Matadero Creek, Adobe Creek and Barron Creek open to the San Francisco Bay.

14.0 UNSTEADY HEC-RAS MODELING

The steady flow HEC-RAS models that were created for each creek were modified to analyze unsteady (time-varying) flow conditions. This allowed for the determination of the outflow hydrographs at each break out location, and therefore a more precise estimate of flood timing and outflow volume than possible with steady flow modeling. Unsteady flow profiles are calculated in HEC-RAS by solving the partial differential equations for continuity and momentum. Several general modifications were made to all the models to allow them to estimate flow leaving the channels and a variety of location-specific modifications were required to achieve model stability.

14.1 LATERAL STRUCTURES

Lateral structures are weirs or gates aligned parallel to the flow that regulate flow into side channels or, in this case, onto the floodplain. Levees and channel banks were modeled as lateral weirs with flood waters passing over them and spilling onto the floodplain. Unsteady HEC-RAS calculates the hydrograph of the flow passing over the structure. The locations and geometries of these structures were determined by: 1) running the 500-yr steady flow HEC-RAS models, (2) reviewing the longitudinal water surface and channel bank profile plots, (3) reviewing the cross section plots, (4) identifying areas where the maximum water surface elevation exceeded the bank elevation, and (5) recording the channel bank elevation and station data along the reach where capacity is likely to be exceeded. Figure 14.1 shows a cross section where the maximum water surface overtops the left bank. Because it is possible that the cross section geometry was simply not extended far enough to capture the entire available flow area, LiDAR and aerial photographs were used to confirm breakout locations. This process was repeated for all models in the SSFBS study area.

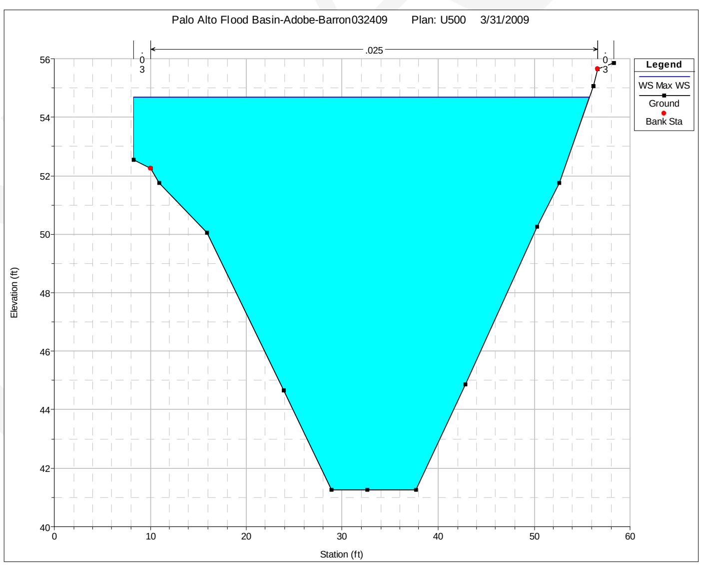


FIGURE 14.1. CROSS SECTION SCHEMATIC SHOWING CHANNEL BANK ELEVATION EXCEEDED BY MAXIMUM WATER SURFACE ELEVATION

A graphical representation of a lateral structure on Adobe Creek is shown in Figure 14.2. With the exception of an undeveloped area at the downstream end of

Calabazas and San Tomas Creeks, the floodplain storage volume is much greater the outflow volume. The floodplains are separated from the channels by floodwalls and levees. slope away from the channel, as evidenced by the 2-dimensional floodplain flow modeling. It was generally assumed that outflows were entering virtually infinite storage areas, i.e. lateral weirs could not become submerged, and outflow would not be limited by floodplain capacity because the storage volume of the floodplains is much greater than the outflow volume, and channels are separated from the floodplains by levees and floodwalls. Although some flow is expected along the length of the structure during the flood event, the "breakout point" to be used in floodplain modeling was located at the lowest point of the structure.

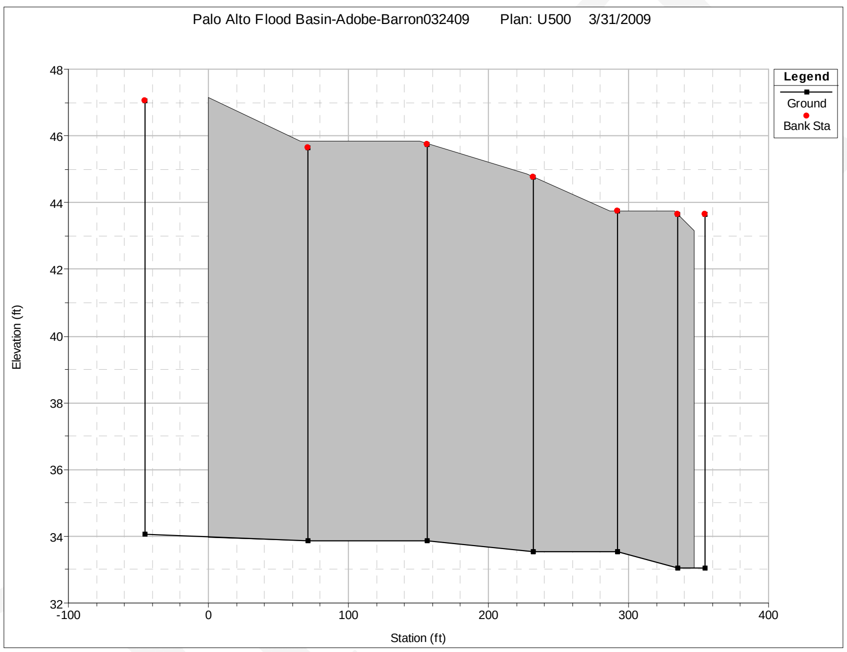


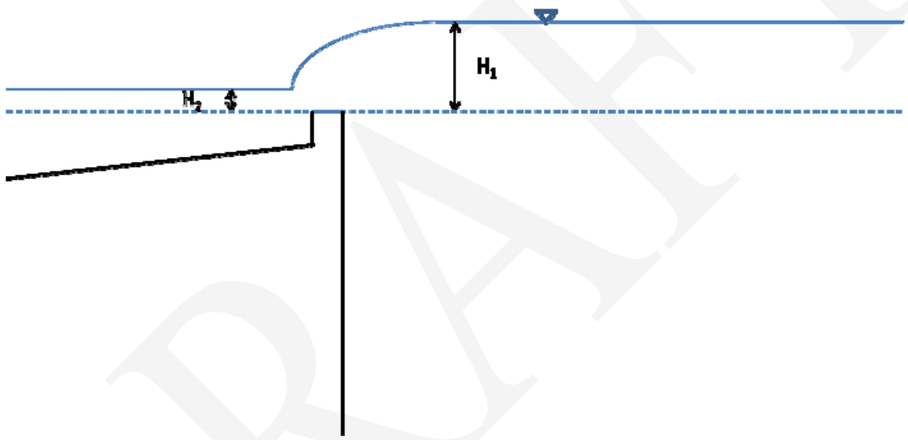
FIGURE 14.2. GRAPHICAL REPRESENTATION OF LATERAL STRUCTURE ON ADOBE CREEK.

Tailwater would limit the amount of lateral flow from the channel when the ratio of the height of the water surface in the floodplain to the height of the water above the weir crest is 0.66 or greater (see schematic).

The assumption that the lateral weirs would not be submerged was tested at several locations by comparing the water surface elevation in the channel simulated in HEC-RAS with the maximum water surface elevation on the adjacent floodplain calculated by FLO-2D. Table 24 presents the modeled water heights relative to the levee crest of four locations in the study. None of the values were above 0.66, and most maximum water surface elevations on the floodplain are below the levee crest, as shown by negative values of H_2.

TABLE 23.1. COMPARISON OF 0.2% PROBABILITY WATER SURFACE HEIGHTS RELATIVE TO LEVEE CREST

Name	River Station	H_1	H_2	H_1/H_2
Adobe Creek	16412	2.5	-1.5	-
Matadero	19200	0.45	0.14	0.3
Guadalupe	38520	0.61	-5.2	-
Stevens	15733	0.65	-0.8	-



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14.2 Inline Structures

Several channels in the study area have drop structures in the upper reaches. The HEC-RAS models received by the District represented these drops with closely spaced cross sections programmed with the longitudinal drop geometry. Because the model is using partial differential equations to calculate the water surface profile, steep drops must be modeled as inline structures, not simply sharp drops in bed profile. To achieve this, inline structures were added to the model geometry by copying the upstream cross section data into the inline structure editor and setting the upstream distance, weir width, and weir coefficient values. Weir coefficients were set to the default value of 2.6.

14.3 Boundary Effects

Additional “dummy” cross sections were added to the geometry files of each creek to reduce unwanted boundary effects at the downstream end. For example, Figure 14.3 shows the longitudinal profile of Permanente Creek, with the dummy cross section added 3000 feet downstream of, and 10 feet below, the surveyed creek geometry.

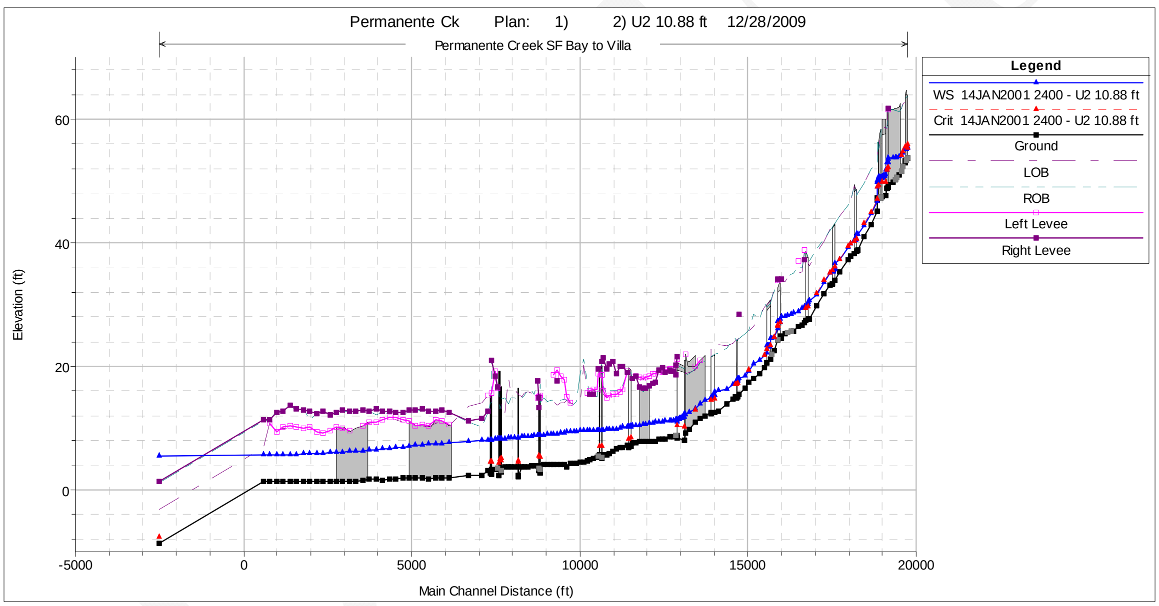

FIGURE 14.3. PROFILE VIEW OF PERMANENTE CREEK SHOWING DUMMY CROSS SECTION ADDED AT DOWNSTREAM END

14.4 Unsteady Flow Data

Unsteady flow files for the 50-, 20-, 10- , 4-, 2-, 1-, 0.4- and 0.2- percent events were created for all models from the corresponding hydrologic analyses described in Section 13 of this document. The hydrologic analyses provided total inflows at the upper end of the study area and total flow rates at junctions. HEC-RAS requires lateral inflows to define the internal boundary conditions. The differences in flows at the upstream ends and flows at the junctions were used to define lateral inflow rates. Figure 14.4 shows an example of lateral inflow, upstream flow, and junction flow hydrographs.

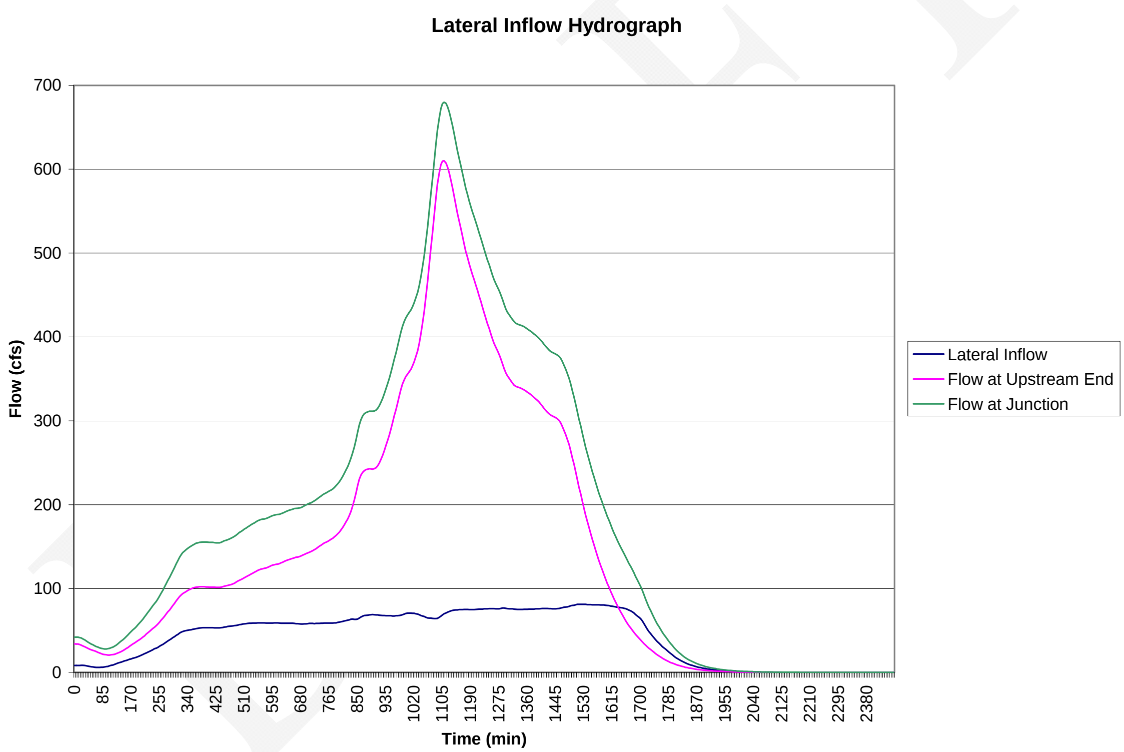

FIGURE 14.4. EXAMPLE OF FLOW AND LATERAL INFLOW HYDROGRAPHS

The flow and lateral inflow hydrographs were repeated seven times to simulate multiple, identical runoff events (Figure 14.5).

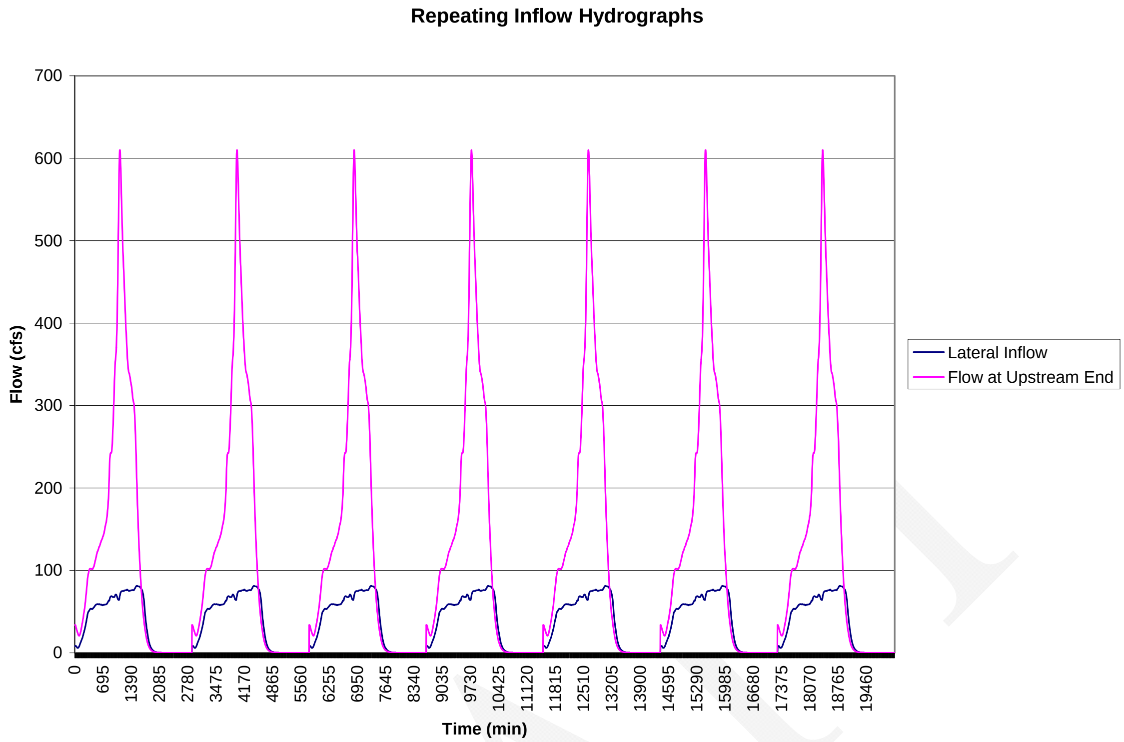

FIGURE 14.5. REPEATING INFLOW HYDROGRAPHS

The hydraulic conditions evaluated by this analysis included a range of downstream boundary conditions representing a combination of tidal action and storm surge. Downstream boundary conditions were defined for all models by creating a quasi-steady water surface elevation file, where the water surface elevation remained constant during the inflow hydrograph, then increased to the next level as the inflow hydrograph repeated. (Figure 14.6).

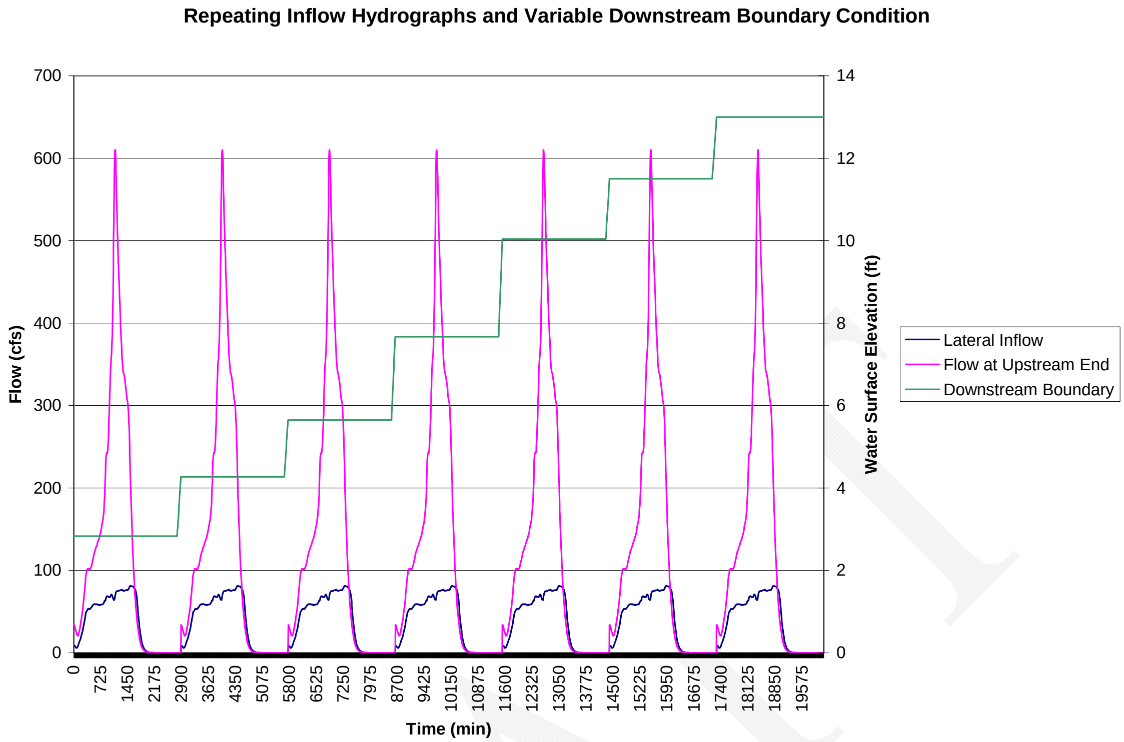

FIGURE 14.6. REPEATING INFLOW HYDROGRAPHS AND VARIABLE DOWNSTREAM BOUNDARY CONDITION

14.5 Unsteady Flow Simulation

Separate plans were created for each return period. Once all of the necessary modifications were performed, the models were run using the full unsteady hydrology described above. Computational time steps were set to a minimum of 6 seconds and a maximum of 20 seconds, as needed to achieve model stability. Simulation durations varied between sites depending on site specific hydrology, such that each complete hydrograph was repeated seven times per plan.

15.0 Coincident Frequency Analysis

Coincident frequency analyses (CFA) based on the predicted water surface elevations for the range of possible downstream boundary conditions were performed to determine an appropriate single downstream boundary condition to be used in the final breakout flow analyses. The range of water surface elevations determined by RAS for a given site was compared to the CFA results. The downstream boundary condition which yielded a water surface elevation most closely matching the CFA result was identified as the appropriate scenario for final breakout hydrograph calculation. In general, predicted water surface elevations were within one-tenth of one foot of each other, regardless of downstream boundary conditions.

15.1 Outflow Hydrographs

Lateral flow hydrographs calculated for each breakout point were exported from HEC-RAS into Excel. The total outflow volumes for each breakout point were calculated by integrating the incremental flow rate over the duration of flow event (Equation 15.1).

$$V = \int_0^t Q dt \qquad \text{(Equation 15.1)}$$

Where:

V = Volume ($feet^3$)

t = Time (sec)

Q = Incremental discharge (cfs)

HEC-RAS will calculate this value automatically, but in this case, each simulation included seven flow events. Therefore, it was necessary to analyze only the hydrograph pertaining to the flood event that was most representative of conditions predicted by the coincident frequency analyses. Flood volumes were used to calculate fluvial contributions to total coastal inundation in the Monte Carlo analysis. Breakout hydrographs were used to model floodplain inundation using FLO-2D.

15.2 Year 0 HEC-RAS Modeling Results

Breakout flows are predicted to occur at many locations in the study area during the largest flow events (i.e. 2-percent return period or greater). Please refer to Table 24 (fold out page) for a complete summary of peak outflow rates and volumes.

Table 24. Summary of Peak Outflow Rates and Volumes

Percent Chance Exceedance	0.2%		0.5%		1%		2%	
Breakout River Station	Peak Outflow (cfs)	Total Outflow Volume (ac.feet)	Peak Outflow (cfs)	Total Outflow Volume (ac.feet)	Peak Outflow (cfs)	Total Outflow Volume (ac.feet)	Peak Outflow (cfs)	Total Outflow Volume (ac.feet)
Adobe (East)								
16399	155	48	144	32	106	13	62	4
18154	95	15	63	8	22	1	0	0
22967	245	36	148	18	48	4	9	0.5
Adobe (West)								
16400	95	15	63	8	22	1	0	0
18155	97	15	65	8	23	2	0	0
23013	690	120	490	67	210	17	43	2
Agua Caliente (North)								
2580	150	7	120	5	0	0	0	0
Agua Caliente (South)								
948	119	11	118	10	86	5	28	1
2643	110	5	83	3	0	0	0	0
Coyote (East)								
77902	67	50	0	0	0	0	0	0
Coyote (West)								
77902	97	78	7	3	0	0	0	0
Fremont (South)								
4700	240	16	130	7	63	3	0	0
Guadalupe (East)								
33200	350	520	7	0.5	0	0	0	0
33894	350	520	7	0.5	0	0	0	0
37240	350	520	7	0.5	0	0	0	0
39602	350	520	7	0.5	0	0	0	0
Guadalupe (West)								
38502	134	190	0	0	0	0	0	0
53570	800	1200	160	40	0	0	0	0
Laguna (East)								
6670	174	21	170	20	124	12	56	4
10950	32	1.6	20	1	0	0	0	0
Laguna (West)								
6671	173	21	169	20	124	12	58	4
11050	109	5	69	3	0	0	0	0
15600	450	30	366	23	100	4	23	0.5
Matadero (East)								
14800	123	17	109	10	0	0	0	0
18340	123	14	125	8	29	1	0	0
19199	207	18	0	0	0	0	0	0
Matadero (West)								
14801	122	17	109	10	0	0		
18341	123	14	125	8	29	1	0	0
19200	207	18	0	0	0	0	0	0
Penitencia (East)								
1600	34	0.5	0	0	0	0	0	0
Penitencia (West)								
1347	40	0.7	0	0	0	0	0	0
4600	420	23	192	5	0	0	0	0
Permanente (West)								
18930	44	28	34	16	21	6	15	5
Stevens (East)								
6200	71	71	73	65	65	50	65	38
6593	16	10	16	9	16	6	16	5
14601	585	460	578	399	436	235	408	139
15733	252	180	232	142	208	101	148	43
16400	363	226	317	168	258	107	130	26
16600	355	218	306	161	246	101	118	25
18100	801	390	585	239	361	101	16	1.2
18225	934	472	699	300	449	144	82	11
18725	451	137	160	34	6	0.6	0	0
Sunnyvale Complex								
Calabazas (East) 1500	1136	425	1022	357	641	170	487	96
Sunnyvale East (East) 1122	975	1282	966	1213	912	1081	852	886
Sunnyvale East (East) 2163	100	98	98	91	89	75	79	56
San Tomas Aquino (West) 34485	2456	3405	2458	3168	2457	2761	2424	2243

16.0 Floodplain Inundation Modeling

Overland flooding in the study area was simulated using computer model FLO-2D (FLO-2D Software Inc 2009). FLO-2D is a quasi 2-d volume conservation model that simulates channel flows and overland flows including unconfined flows over complex topography and roughness, split channel flows, and urban flooding. The model numerically routes a flood hydrograph while predicting the area of inundation and floodwave attenuation. FLO-2D uses the full dynamic wave momentum equation and a central finite difference routing scheme with eight potential flow directions to predict the progression of a flood hydrograph over a system of square grid elements.

16.1 Surface Topography

FLO-2D requires two sets of data for any flood simulation, a digital terrain model (DTM) and either an inflow hydrograph or a discretized rainfall event. The potential flow surface topography is represented by a square grid system. The grid elements (or grid cells) are assigned elevations from an interpolation of the DTM points. A processor program GDS (grid developer system) generates the grid system and assigns the elevations. The GDS superimposes a grid system over the DTM points and interpolates grid cell elevations using DTM point filters. It automatically generates the FLO-2D floodplain and other data files to start an overland flood simulation. Images can be imported to the GDS to assist graphical editing. Any size grid cell can be used in the model, but the computational time step is governed by wave celerity and small grid cells will require small time steps and long model run times. A typical square grid cell size will range from 10 feet to 500 feet. The number of grid cells is unlimited. However, if the number of grid cells exceeds 100,000, model simulation may be very slow and may take days and weeks.

The coordinate system reference for the SSFBSS FLO-2D models is the California State Plane NAD 83, Zone 3 (0403), in US Survey Feet, as the horizontal coordinate system. The vertical datum is NAVD 88.   All model boundary conditions and output will be referenced to this datum.

16.2 Surface Roughness

For flow less than 0.2 feet deep (where the flow depth is on the order of the roughness element), a default value of shallow flow Manning's roughness coefficient of 0.2 is used in FLO-2D. For flow depth between 0.2 and 0.5 feet, the shallow flow roughness coefficient is reduced by half. To improve the timing of the floodwave progression through the grid system, a depth variable roughness is assigned for flow depths ranging from 0.5 feet to 3 feet. For depth in excess of 3 feet, a user-defined Manning's roughness value is used.

16.3 Inflow Hydrographs or Rainfall

Inflow hydrographs can be designated for either channel or floodplain nodes. The number of inflow hydrograph nodes is unlimited. FLO-2D can also perform as hydrologic model and spatially variable rainfall data can be assigned for depth area reduction or to simulate a moving storm. The rainfall is routed as overland sheet flow or as rill and gully flow until it is intercepted by a main channel. The flood routing can continue in the river channel in the same model creating a combined hydrologic and hydraulic model.

16.4 Routing Algorithm Stability and Volume Conservation

Computational time steps are determined by FLO-2D and typically range from 1 to 30 seconds. The time step is incremented and decremented according to strict flood routing numerical stability criteria. Numerical stability is linked to volume conservation. The key to any successful flood routing model is volume conservation. When the model accurately conserves volume the model runs faster. Volume conservation is tracked and is reported both during the simulation and in summary output files.

16.5 Buildings and Flow Obstructions

Floodplain storage loss due to buildings, topography or even large trees on a grid cell basis can be incorporated into a flood model using area reduction factors. A portion of a grid cell or the entire cell can be removed from potential inundation during the flood simulation. Reduced flood storage forces more flow downstream. The flow exchange between grid cells can be partially or entirely obstructed with a flow width reduction factor for any or all of the eight flow directions.

16.6 Model Output, Results and Mapping

The floodwave progression over the flow surface can be viewed along with a plot of the inflow hydrograph while the model is running. The main output results from a flood simulation include maximum water surface elevation for each grid cell within the computational domain, maximum flow depth and velocity, and flow depth and velocity at the end of the simulation. The simulation results can be viewed graphically in the MAPPER post-processor program. MAPPER automatically generates and saves shape files of flood plots for viewing in ArcGIS.

16.7 Data Sources

The following data were used to develop 2-d models of the floodplain areas along the study streams and to simulate overland flooding: (1) 3 feet by 3 feet grid DTMs covering the study area within Alameda County and 5 feet by 5 feet grid DTMs within Santa Clara County; (2) aerial imagery of the study area; (3) flow breakout locations; (4) breakout flow hydrographs; and (5) highway middle barrier data. The flow breakout locations are shown on the inundation maps attached in the appendix of this report. Breakout flow characteristics are summarized in Table 23. The District and the contractor (Northwest Hydraulic Consultants) used this data to develop 2-d models and to simulate propagation of the breakout flows through the study area.

16.8 Modeling Approach

The main assumption in the FLO-2D model development was that only one stream can be flooded at a time. The only exception was Sunnyvale Complex simultaneous flooding on all the three streams comprising the complex (Sunnyvale East Channel, Calabazas Creek, and San Tomas Aquino Creek) was simulated. It was also assumed that all breakout flows happen simultaneously and that no overland flow returns back to the flooded creek channel. Creek channels and levees running along the creeks could not be accurately reproduced in the 2-d model topography because of their small size. Therefore, creek channels were excluded from the models by setting model boundaries along creek banks/levees.

A total of eighteen 2-d models were developed. Generally, separate models were developed for left and right floodplains due to the large size of the flood prone areas. A single model was developed for Sunnyvale Complex to combine all the streams within this area. A single model combining both left and right floodplains was developed for Lower Penitencia Creek. Model computational domains included anticipated inundation areas and usually extended from the most upstream breakout locations all the way down to the bay. If subsequent model runs indicated that the simulated inundation area was limited by the computational domain (i.e. simulated flow hit the model boundary and water surface elevation at this location exceeded local ground elevation), the computational domain was increased and model run was repeated. Land areas included in the final versions of the models ranged from 1,610 to 2,650 acres. The models developed and model main parameters are summarized in Table 25.

The grid resolution used for the models varied from 30 feet to 40 feet, depending on the size of the computational domain. An average ground elevation within each grid cell was computed from the DTM data. The grid cell sizes used were sufficiently small to represent relatively small-scale topographic details (such as streets and highways) and at the same time provided manageable run times. The number of grid cells in the models ranged from 56,821 to 127,158.

Table 25. FLO-2D models main parameters.

Stream	Modeled floodplain	Model area (acres)	Grid cell size (feet)	No. of grid cells
Adobe Creek	Left	580	40	15852
Adobe Creek	Right	1250	40	34047
Agua Caliente	Left	850	40	23242
Coyote Creek	Left	2,180	30	103,862
Coyote Creek	Right	1,950	30	92,050
Fremont	Left	520	20	56985
Guadalupe River	Left	2,620	40	70,113
Guadalupe River	Right	2,650	30	127,158
Laguna and Agua Caliente	Left & Right	520	40	14280
Laguna Creek	Right	680	40	18500
Lower Penitencia Creek	Left & right	1,200	30	56,821
Matadero Creek	Left	840	40	22946
Matadero Creek	Right	580	40	15742
Permanente	Left	1250	40	34047
Stevens Creek	Right	2,300	30	109,673
Stevens Creek	Left	1,610	30	76,786
Sunnyvale Complex	Left & right	1,290	30	61,611
Scott Creek	Left	96	20	10450
Scott Creek	Right	262	20	28530
Scott Creek	Basin	150	20	16335

A global Manning's roughness coefficient of 0.04 was assigned to all grid cells in all the models. This value was within the 0.02-0.05 range suggested in the FLO-2D user's manual for asphalt and concrete and within the 0.04-0.10 range suggested for grassland and open floodplain areas. The value of 0.04 was believed to be generally representative of the urbanized areas modeled. No local site-specific roughness values were used because of the absence of relevant information.

In order to include storage capacity loss due to buildings and other structures, an Area Reduction Factor (ARF) was assigned to each grid cell. In each modeled area sub-regions with similar land types were outlined based on visual assessment of aerial photographs and appropriate ARF values were assigned to all grid cells within the selected sub-regions. Land types and corresponding ARF values used in this study are summarized in Table 26. ARF sub-regions were specified not to block major highways and streets. Most highways and streets appeared in the 2-d models as topographical depressions and were important conveyors of overland flows.

Table 26. Area Reduction Factors for different land types for use in FLO-2D model (adopted from USACE 2007 and NHC 2008).

Land use type	Area Reduction Factor (ARF)	Land use type	Area Reduction Factor (ARF)
Dense residential	0.4	Commercial	0.5
Open residential	0.2	Rural / agricultural	0
Downtown	0.7	Dense vegetation	0
Park areas	0	Default floodplain value	0
Industrial	0.6	Large individual buildings	1

Due to the very large size of the study area, highly complex character of the urban environment, complex flooding pattern (which was often difficult to anticipate), absence of relevant data, and time and budget constraints, no width reduction factors were specified and no individual street, levee, or hydraulic structure (such as bridges, culverts, drainage ditches, highway noise walls, etc) were modeled explicitly. Overland flow in the models was entirely governed by the land surface topography as contained in the DTMs. Due to averaging of surface topography within model grid cells (30x30 feet or 40x40 feet square elements), there could be areas where narrow dikes and roads were smoothed or erased in the models. However, it is believed that such small-scale features did not affect overall flooding patterns. The only additional feature which was included in the models was a 3.5 feet high concrete middle barrier running along portions of Highways 101, 237, and 880. This middle barrier was specified on a grid cell basis by rising the grid cell elevation in the model by 3.5 feet.

Inflow points and corresponding inflow hydrographs were specified at the breakout locations. Outflow points were assigned along significant dry channels to allow spillage of overland waters into these channels. To simulate flow along the highway middle barriers, outflow points were also specified at the locations where these barriers crossed model boundaries. Westward outflow data simulated in the Stevens Creek right floodplain model for the Highway 101 Bridge were used to specify inflow hydrographs for the Stevens Creek left floodplain model (Table 5). In most cases, floods flow toward the Bay. Levees between the salt ponds and the urban areas prevent flood flow from reaching the Bay. Flows reaching the levees pond behind the levee and in some cases overtop the levee. If overtopping of the most downstream levee was expected (which was the case for Sunnyvale Complex), model grid cells along this levee were assigned actual levee elevations (most levees due to their narrow width were usually smoothed in the 2-d models) and outflow points were specified along the levee crest to simulate spillage of flood waters into the salt ponds.

The developed models were not calibrated due to the absence of relevant calibration data. The models were used to route flood flows from the breakout locations through the urban areas as they progressed towards the San Francisco Bay. The simulation time step in the model runs was variable and was adjusted by FLO-2D automatically in order to provide numerically stable solutions. The simulation time was set to 72 hrs from the beginning of the flood for all the models. This simulation time was sufficiently long so that the breakout flows reached the downstream model extent or accumulated in local depressions and no significant water flow occurred at the end of the simulations. Model run times generally ranged from a few hours to a few days. For some big models (such as the Guadalupe River model), simulations lasted for over a week.

16.8.1 Sensitivity Analyses

FLO-2D relies on a number of user-defined input parameters to perform hydrodynamic computations. Recommended value ranges are available for these parameters, but selection of the final value is dependent on the specific application and the modeler's judgment. The validity of the selected values is usually checked by comparing model results with measured flow data. If necessary, the model parameters are then adjusted to obtain the best agreement between the modeled and measured data. This process of adjusting model input parameters is called "model calibration". However, no detailed flood inundation calibration data are available for the study area. Sensitivity analyses were therefore performed on a range of modeling parameters to evaluate the reasonableness of the model results. The parameters tested included: (1) Manning's roughness coefficient, (2) Area Reduction Factors, and (3) model grid cell size. Sensitivity analyses consisted of changing selected modeling parameters (while keeping other modeling parameters unchanged) and assessing the change in the simulated results.

The sensitivity analyses were conducted using the Coyote Creek left floodplain model as an example. This model had a moderate inundation area and relatively short run times, which allowed testing different modeling parameters within reasonable time limits. Variations in results due to the change in modeling parameters were assessed by comparing computed inundated areas, inundation depths, and overall flooding patterns. Given the similar topographic and hydraulic conditions, results of sensitivity tests conducted for the Coyote Creek left floodplain model are considered to be representative of the other basins modeled in this study. The sensitivity runs were conducted for the 0.2% chance exceedance event. Results of the sensitivity runs are presented in Table 27 and briefly discussed below.

Table 27. Results of model sensitivity runs for Coyote Creek (left floodplain) for 0.2% Exceedance Event.

Model parameters			Simulation results			
Grid cell size (feet)	**Manning's roughness coefficient**	**Area Reduction Factor**	**Maximum inundated area (acres)**	**Area change** (%)**	**Mean (max) water depth, (feet)**	**Mean (max) depth change** (%)**
Main model						
30	0.04	Yes	88.6	--	0.695 (4.74)	--
Sensitivity test models						
30	0.02*	Yes	84.3	-4.9	0.705 (4.78)	1.4 (0.8)
30	0.02*	No*	72.8	-17.8	0.71 (4.3)	2.3 (-9.3)
30	0.04	No*	77.3	-12.8	0.695 (4.29)	0.0 (-9.5)
30	0.06*	Yes	92.0	+3.7	0.691 (4.7)	-0.5 (-0.8)
30	0.06*	No*	80.1	-9.6	0.696 (4.2)	0.1 (-11)
40*	0.04	Yes	89.6	+0.3	0.695 (4.55)	0.0 (-4.0)

* Changed parameter; ** Relative to main model results.

16.8.2 Manning's Roughness Coefficient

A single representative value of Manning's roughness coefficient of 0.04 was used in the main model. It is known, however, that this parameter may vary within quite significant limits depending on many local factors which are difficult to define a priori without direct stream flow measurements. To determine the effect of changing surface roughness coefficient on simulated flooding characteristics, sensitivity runs were performed using roughness coefficients of 0.02 and 0.06. The Manning's coefficient of 0.02 corresponds to simple, plane surface conditions, while the coefficient of 0.06 is close to the generic value for overland flow suggested in the FLO-2D user's manual.

The sensitivity runs show that reduction of the Manning's roughness coefficient from 0.02 to 0.06 results in minor changes in predicted inundated area and flood depths (Table 27). The computed overall flooding pattern does not change noticeably when varying surface roughness. The largest changes are obtained for the maximum depth, while very little changes are simulated for the inundation area and mean inundation depth. On the whole, given the complex surface topography and heavily urbanized character of the model area, differences in the modeling results caused by varying surface roughness appear to be acceptably minor.

16.8.3 Area Reduction Factor

To determine the effect of Area Reduction Factors (ARFs) on the computed flooding pattern and overland flow characteristics, a test run was conducted with the ARF value set to zero for all grid cells within the computational domain (i.e. flooding was simulated ignoring the effect of building on floodplain storage loss). The results of this test run indicate that without ARFs the simulated inundation areas and maximum depths decrease, while mean depths increase. Due to the greater floodplain storage capacity without ARFs (i.e. without buildings), a greater attenuation of the floodwave occurs in the upper reaches and a smaller volume of water reaches the downstream end of the basin. In the Coyote Creek basin, overland flood waters mainly propagate downstream via streets, with little shallow sheet flow through built-up areas. This is why changes caused by ignoring the effect of building appear to be relatively minor. However, in areas with substantial overland sheet flows (such as along Stevens Creek and the Guadalupe River), these changes could be more pronounced.

16.8.4 Grid Cell Size

The model grid cell size for the main modeling run was set to 30 feet. To determine the effect of the gird cell size on computed flooding characteristics, a sensitivity run was conducted using a 40 feet model grid. Analysis of the sensitivity test results revealed that an increase of the grid cell size from 30 feet to 40 feet causes a 0.3% increase in the inundation area (from 389 acres to 390 acres), and minor changes in predicted mean and maximum flood depths (Table 27). The overall flooding pattern remains unchanged. Although using smaller model grid sizes generally produces more accurate predictions (due to better resolution of the bed surface topography), the differences in major flood characteristics between the finer 30 feet grid model and the coarser 40 feet grid model appear to be insignificant.

An attempt was made to run a 20 feet model of the Coyote Creek left floodplain. However, after two weeks of simulations, the test run was terminated. Use of the grid cell size smaller than 30 feet appears to be impractical from the point of view of manageable model run time. Therefore, use of the 30 feet to 40 feet grid models of the study floodplain areas appears to be the best balance between modeling details and efficiency.

17.0 Year 0 Without Project Results

The results of the year 0 HEC-RAS and FLO-2D analyses are discussed in the following sections for each of the streams. The resulting floodplains as determined by the FLO-2D modeling effort are presented in Plates 47-76.

Flood maximum inundation depths predicted by the FLO-2D models were mapped in GIS. One inundation map per stream was produced and combined flow data simulated for both (left and right) floodplains. Simulated maximum inundated areas and inundation depths are summarized in Tables 28 and 29. Below is a brief discussion of overall flooding pattern simulated for each stream.

17.1 Adobe Creek

Breakout flows are expected to occur at six locations on the east and west sides of Adobe Creek. All flows trend generally northwest toward San Francisco Bay, but none are expected to cross Highway 101, assuming the continued presence of a 3 feet high median barrier at year 0. Two breakout points, one on either side of the channel, are simulated approximately 700 feet upstream from Alma St., two are just upstream of Middlefield Rd., and two are just upstream of

Louis Rd. Some ponding is expected upstream of Alma St. and Middlefield Rd., but the most significantly inundated areas are within 0.5 miles upstream of Highway 101 in the area bordered generally by Barron Creek, Highway 101 and San Antonio Rd. Adobe Creek inundation depth maps for the 1-, 0.5-, and 0.2-percent chance exceedance events are presented in Plates 47 through 49.

The simulated maximum inundated areas for Adobe Creek were 218, 189 and 83 acres for the 1-, 0.5-, and 0.2-percent chance exceedance events, respectively. The mean inundation depth was approximately 1 foot for all events, and the maximum depth was nearly 14 feet.

17.2 Barron Creek

No flooding is predicted to occur on Barron Creek, although it is part of the Matadero and Adobe Creek drainage systems. Flooding in the vicinity of Barron Creek is expected to occur from those two channels.

17.3 Coyote Creek

Flood flows from Coyote Creek spill into both the left (west) and right (east) floodplains. All the flow breakout locations are concentrated downstream from Interstate 880 in the vicinity of Charcot Avenue. Overland flows occur in wide bands through predominantly commercial and industrial areas.

On the left floodplain, the ground surface slopes away from Coyote Creek toward the Guadalupe River. As a result, overland flows travel westerly and then northwesterly away from the creek. On the right floodplain, overland flows travel north between the Coyote Creek channel and Interstate 880.

17.4 Fremont Creek

Overbank flows from Fremont Creek occur at one location: on the south side of the channel, about 1000 feet downstream of Warm Springs Rd. Flow travels predominantly west, across Milmont Dr, toward Highway 880, where it is eventually blocked by the elevated roadway and center median barrier. The 1% chance exceedance outflow is minimal, resulting in sheet flow and inundation of topographic depressions in the area just east of Milmont Dr., with some flow continuing down an access way to Kato Rd.

The simulated maximum inundation areas for the 0.2-, 0.5-, and 1-percent chance exceedance flow events were 34, 20, and 8 acres, respectively. Mean depths were approximately 1 foot for all events, and the maximum simulated depths were approximately 4 feet for all events. Fremont Creek inundation depth maps for the 0.2-, 0.5-, and 1-percent chance exceedance flow events are presented in Plates 52 through 54.

17.5 Guadalupe River

According to the FLO-2D results, the 0.2-percent chance exceedance event will cause widespread overland inundation on both left and right floodplains along the lower Guadalupe River. Overbank outflows from the river into the left (west) floodplain occur at two locations and into the right (east) floodplain at four locations. Left-side breakouts are located at San Jose International Airport and downstream of Montague Expressway. Right-side breakouts are all located between Montague Expressway and Tasman Drive.

On the left floodplain, flows from the downstream-most breakout location travel in the north-eastern direction through a network of streets and inundate large areas between Lick Mill Park and Highway 237. Some waters flow through openings underneath Highway 237 (via Lafayette Street and Great America Parkway) and accumulate in topographic depressions north of the highway. Overland flows from the upstream breakout at San Jose International Airport propagate north-westerly, fill ground depressions in the airport area, pond at Highway 101, spill over the highway at a few locations, travel north through a network of streets and as a sheet flow through built-up areas, and then generally follow the path of the flows from the downstream breakout. Flooded areas include northern part of San Jose International Airport, residential and commercial areas generally located between the Guadalupe River and Lafayette

Street, as well as commercial and open areas in the vicinity of Highway 237. No water spills into the Baylands. All the overland flood waters are contained within local storage areas. During the 0.2-percent chance exceedance event, the maximum inundated area on the left floodplain is 739 acres, the mean inundation depth is 2.05 feet, and the maximum inundation depth is over 21 feet.

On the right floodplain, overland waters from the 0.2-percent chance exceedance event are conveyed north and north-west through commercial and residential areas, pond at Highway 237, spill over the highway between 1st Street and Zanker Road, inundate a vast area north of Highway 237, and pond behind high levees surrounding salt ponds. No water spills into the Baylands. The maximum area of inundation on the right floodplain is 1,233 acres, the mean inundation depth is 2.14 feet, and the maximum inundation depth is over 13 feet. The total inundated area (including both the left and right floodplains) is 1,972 acres.

The 0.5-percent chance exceedance event causes localized flooding on the left floodplain between the breakout location at the airport and Highway 101. The maximum overland inundation area is 42 acres, the mean inundation depth is 1.54 feet, and the maximum inundation depth is almost 20 feet. No water spills over Highway 101. All flood waters accumulate within local storage areas. Guadalupe River inundation depth maps for the 0.5- and 0.2-percent chance exceedance event are presented in Plates 55, 56A and 56B. Plate 56 was split across two pages due to the large extent of flooding simulated for that area.

17.6 Laguna and Agua Caliente Creeks

The majority of the flooding on this system is expected to occur to the west side of Laguna Creek, between Auto Mall Parkway and Highway 880, with some significant flooding also predicted just upstream of the confluence of Laguna and Agua Caliente Creeks, and on the south side of Agua Caliente Creek downstream of Highway 880. Simulated flows are completely blocked by the 3 feet high center median barrier on Highway 880.

The total simulated maximum inundation areas for both creeks are 156, 105, and 56 acres for the 500-, 250-, and 100-yr events, respectively. Maximum simulated depths were all in the Laguna Creek floodplain, which reached depths of 12, 11, and 3 feet for the 500-, 250-, and 100-yr events, respectively. Laguna and Agua Caliente Creek inundation depth maps for the 1-, 0.5-, and 0.2-percent chance exceedance event are presented in Plates 57 through 59.

17.7 Lower Penitencia Creek

According to the 2-d model results, the simulated 0.5- and 0.2-percent chance exceedance event do not cause significant overland flooding along Lower Penitencia Creek. Inundated areas are limited to topographic depressions (mainly streets) in the vicinity of the flow breakout locations. The maximum inundated area is 17 acres for the 0.2% event and about 7 acres for the 0.5% event. The mean inundation depth is 1.58 feet and 1.11 feet, respectively. The maximum inundation depth is over 4 feet for the 0.2% event and over 2 feet for the 0.5% event. Lower Penitencia Creek inundation depth maps for the 0.5- and 0.2-percent chance exceedance event are presented in Plates 60 and 61.

17.8 Matadero Creek

Significant flooding is predicted to occur in the Matadero Creek area in response to the 0.5- and 0.2-percent chance exceedance events precipitation events. Flow is predicted to leave the channel at six general locations, four upstream of Alma St and two upstream of Middlefield Rd. Four breakout flows occur just upstream of Alma St:, one on each side of the channel approximately 900 feet upstream of the roadway, and one on each side of the channel approximately 150 feet upstream of the roadway. Two breakout flows, one of each side of the channel approximately 500 feet upstream of Middlefield Rd, continue across Middlefield Rd, eventually inundating much of the area bounded by Highway 101, Colorado Ave, and the Oregon Expressway. Predominately shallow flooding (less than 1 feet) was simulated on the east side of the Matadero Creek channel, mostly located between the channel and Loma Verde Ave.

The maximum inundated area for Matadero Creek is 220 acres for the 0.2% and 118 acres for the 0.5% event. 1% event breakout flows are predicted to occur only at the two locations approximately 150 feet upstream of Alma St., with inundated area limited to 5 acres, all to the south and west side of Alma St. The mean flood depth is 0.7 feet for the 0.2% event, 0.6 feet for the 0.5%- and 1% events. Maximum inundation depths are expected to range from 10 to 8 feet for the design storm events. Matadero Creek inundation depth maps for the 1-, 0.5-, and 0.2-percent chance events are presented in Plates 62 through 64.

17.9 Permanente Creek

Flooding is predicted to occur just downstream of the Central Expressway, to the west side of the Permanente Creek channel. Capacity limitations upstream of the study area limit the occurrence of breakout flows up to this point, where excess flows are added to the system. Peak outflows range from 44 cfs during the 0.2% event to 15 cfs during the 1% event. Flows travel generally northward from the breakout point, flooding into the neighborhood bordered by the channel, Rengstorff Ave and Middlefield Rd., filling in topographic depressions in that area. Flows continue as shallow overland flow north across Middlefield Rd. inundating small topographic depressions to the north, with a portion of the flow eventually reaching Highway 101.

Maximum inundation areas for the 0.2%-, 0.5%-, and 1% flow events were 109, 71, and 36 acres, respectively. Mean flood depths were less than 0.4 feet for all flood events, with maximum depths from approximately 3 to 4 feet. Permanente Creek inundation depth maps for the 0.2%-, 0.5%-, and 1% flow events are presented in Plates 65 through 67.

17.10 Scott Creek

Hydraulic modeling predicts breakout flows to occur at four locations: upstream of the SVRT railroad tracks to the south, upstream of Milmont Drive to both the north and south, and just upstream of I-880, before flowing into a large storage area downstream of I-880. The storage area downstream is being developed as part of the Bayside Business Park (Phase II). Future flows will be split between a detention basin on the south end of the lot and a bypass channel consisting of five 9’ x 5’ reinforced concrete boxes. Breakout flows that exit the channel upstream of I-880 and flow into the storage area are contained by said storage area, even during the 0.2% event. Flows exiting the north and south of the channel at Milmont Dr. appear to be generally contained by the existing roadway topography, not extending beyond Kato road to the north or Dixon Landing Rd to the south.

Flood waters are predicted to cross into Santa Clara County from Alameda County from both breakout points on the south side of the channel: Milmont Drive and upstream of the SVRT tracks.

17.11 Stevens Creek

According to the FLO-2D model results, the 0.2%-, 0.5%-, and 1% flow events will cause widespread sheet flooding along Stevens Creek. Overall flooding pattern is generally similar for these events. The modeled overbank flow breakouts from the creek flow into the right-side (east) floodplain. Seven breakout points are located between Central Expressway and Highway 101 and two breakout points are located at the northern extent of Moffett Airfield in the vicinity of Baylands. Overland flows from the upstream breakouts travel generally in a northerly direction along the streets and as a sheet flow through residential areas to Highway 101. At Highway 101, flood waters pond in front of the middle barrier and then flow in both easterly and westerly directions. Easterly flows reach Ellis Street and then turn north underneath Highway 101 and disperse into Moffett Airfield. Westerly flows cross the Highway 101 Bridge and then travel along the highway further west (generally until Permanente Creek) and partially spill over the middle barrier and flow north between leveed Stevens Creek and Shoreline Boulevard.

Some flood water ponds on the right floodplain at Highway 101, overtops the middle barrier, and flows north through the Ames Research Center and Moffett Airfield. Eventually, most of the overland flood water flows through the right floodplain of the creek and accumulate in storage areas located north of Moffett Airfield and separated from the Baylands by a high levee. Flood waters from the two downstream most breakouts flow directly into this storage area. Some waters accumulated at the northern end of the airfield and spill into Moffett Field Gold Course. On the left-side floodplain of Stevens Creek, overland flood waters fill local depressions and spill into Permanente Creek at Highway 101.

Modeling results indicate that during the 0.2- and 0.5-percent chance exceedance event flood waters ponded at the northern end of Moffett Airfield and in the northern part of Moffett Field Golf Course may start spilling over low portions of the outboard levee into adjacent salt ponds and into the channel network surrounding the golf course. Therefore, model predictions for the maximum water depths at this location may be conservative and may overestimate local depths in this area by 0.2-0.3 feet. However, given that this is a non-residential area dominated by coastal flooding, such over-prediction was considered insignificant and are within the accuracy of the modeling approach.

During the 2% flood event, overland flows from upstream breakouts travel north, pond at the Highway 101's middle barrier, and are conveyed easterly and westerly along Highway 101. No overtopping of the middle barrier occurs during the 2% flood. Easterly flows reach Ellis Street, then run north underneath Highway 101, inundate part of Moffett Airfield, and eventually accumulate in the floodplain storage area at the downstream end of the airfield. Outflows from the downstream breakouts also accumulate in this storage area.

17.12 Sunnyvale Complex

All the overbank breakout flows from the creeks comprising Sunnyvale Complex (Sunnyvale East Channel, Calabazas Creek, and San Tomas Aquino Creek) occur into two floodplain storage areas located at the confluence of the creeks. The storage areas are bordered by levees from the west, north, and east and Highway 237 from the south. Model simulations indicate that the volume of the 52-acre storage area at the confluence of Calabazas and San Tomas Aquino Creeks is completely filled with breakout waters for all the modeled floods (0.2-, 0.5-, and 1-percent events). Storage capacity of this floodplain area is limited by the elevation of the north levee (approximately 12.8 feet). This levee is significantly lower than Highway 237 that borders the area from the south (minimum elevation of the highway is about 20.1 feet plus 3.5 feet high middle barrier). Therefore, after filling the storage volume, this floodplain area will no longer be able to accept any inflows, and all the flood waters in excess of the creek channels capacity will start spilling north into salt ponds (which due to their very large size were assumed to have unlimited storage capacity for modeling purposes). Peak inflows into this storage area during the simulated flood events range from 642 cfs to 1,148 cfs for Calabazas Creek and from 2,457 to 2,459 cfs for San Tomas Aquino Creek. However, the breakout hydrographs employed in the FLO-2D models were developed on the assumption of unlimited floodplain storage capacity in the HEC-RAS model. Therefore, inflow into the floodplain storage area at the confluence of Calabazas and San Tomas Aquino Creeks is overestimated. This results in prediction of unrealistically high water levels in this storage area in the FLO-2D model (over a dozen of feet above the north levee), even though outflow from this storage area over the outboard levee into baylands was provided in the model. Since the water level in this storage area can not be higher then the water surface elevation in the adjacent creeks and can not exceed the elevation of the north levee (all waters in excess of this elevation will spill north into adjacent salt ponds), water depths in this floodplain storage area were mapped relative to the elevation of the north levee of 12.8 feet.

Storage capacity of the floodplain area at the confluence of Sunnyvale East Channel and Calabazas Creek appears to be sufficient to accommodate all the inflows from these creeks for all the simulated flood events. No overtopping of the levees or Highway 237 surrounding this storage area occurs. Examination of the modeling results revealed that the maximum inundation level simulated for this floodplain area is actually below the elevation of the levee surrounding a 23-acre pond at the north-east corner of this area (next to Calabazas Creek). The levee separating this pond from the rest of the area is very narrow and therefore was significantly smoothed in the model. Distribution of the volume of water accumulated in this pond in the model over the rest of the inundated area results in a 0.2-0.3 feet increase in the simulated maximum depths. Given the relatively large size of this pond, the pond was excluded from the inundation area in the mapped data, and the mapped maximum depths were adjusted accordingly. The inundated area at the confluence of Sunnyvale East Channel and Calabazas Creek (adjusted for the non-flooded 23-acre pond at Calabazas Creek) is 248 acres for the 0.2% flood, 246 acres for the 0.5% flood, and 240 acres for the 1% event. The mean inundation depth is 4.92 feet, 4.68 feet, and 4.22 feet, respectively. The maximum inundation depth is over 10 feet for the 0.2% and 0.5% flood events and over 9 feet for the 1% flood. The total inundated area simulated for Sunnyvale Complex (including both floodplain storage areas) is 300 acres for the 0.2% flood, 298 acres for the 250-year flood, and 292 acres for the 1% flood. Sunnyvale Complex inundation depth maps for the 1-, 0.5-, and 0.2-percent exceedance events are presented in Plates 71 through 73.

Table 28. Simulated Maximum Inundated Area (acres)

Stream	Floodplain	Percent Chance Exceedance					
		0.2%	0.4%	1%	2%	4%	10%
Adobe Creek	Left	218	189	83		--	
Adobe Creek	Right	134	101	48		--	
Agua Caliente	Left	24	22	15		--	
Coyote Creek	Left	89	--	--		--	
Coyote Creek	Right	52	--	--		--	
Fremont	Left	34	20	8		--	
Guadalupe River	Left	739	42	--		--	
Guadalupe River	Right	1,233	0	--		--	
Laguna and Agua Caliente	Left & Right	24	22	12		--	
Laguna Creek	Right	68	61	29		--	
Lower Penitencia Creek	Left & right	17	7	--		--	
Matadero Creek	Left	115	68	2		--	
Matadero Creek	Right	105	50	3		--	
Permanente	Left	109	71	36		--	
Scott Creek	Left	40	26	18			
Scott Creek	Right	6	1.5	1.0			
Scott Creek (DS I-880)	Basin	104	97	96			
Stevens Creek	Right	1,080	972	682	431	175	15
Stevens Creek	Left	162	124	61	41	0	0
Sunnyvale Complex	Left & right	300*	298*	292*		--	

* Adjusted for non-flooded 23-acre pond on west side of Calabazas Creek (see the text of the report for explanation

Table 29. Simulated Mean and Maximum (in parentheses) Inundation Depths (feet)

Stream	Floodplain	Percent Chance Exceedance					
		0.2%	0.4%	1%	2%	4%	10%
Adobe Creek	Left	1.0 (13.9)	0.71 (13.8)	0.2 (13.5)	--	--	--
Adobe Creek	Right	1.0 (5.9)	0.6 (5.3)	0.6 (3.7)	--	--	--
Agua Caliente	Left	0.9 (5.3)	0.9 (5.1)	0.6 (2.8)	--	--	--
Coyote Creek	Left	0.695 (4.74)	--	--	--	--	--
Coyote Creek	Right	0.76 (4.73)	--	--	--	--	--
Fremont	Left	0.8 (4.3)	0.6 (4.1)	0.5 (3.8)	--	--	--

Guadalupe River	Left	2.05 (21.1)	1.54 (19.9)	--	--	--	--
Guadalupe River	Right	2.14 (13.2)	--	--	--	--	--
Laguna and Agua Caliente	Left & Right	1.4 (4.9)	1.4 (4.7)	1.3 (4.4)	--	--	--
Laguna Creek	Right	1.3 (11.5)	1.2 (10.9)	0.9 (3.6)	--	--	--
Lower Penitencia Creek	Left & Right	1.58 (4.87)	1.11 (2.39)	--	--	--	--
Matadero Creek	Left	0.65 (10.18)	0.53 (9.94)	0.86 (8.14)	--	--	--
Matadero Creek	Right	0.66 (5.0)	0.61 (4.3)	0.47 (1.7)	--	--	--
Permanente	Left	0.36 (3.54)	0.27 (3.36)	0.24 (2.6)	--	--	--
Scott Creek	Left	0.6 (6.5)	0.4 (4.5)	0.1 (3.4)			
Scott Creek	Right	0.1 (5.8)	0.04 (4.4)	0.01 (3.0)			
Scott Creek	Basin	4.6 (1.5)	1.1 (4.1)	1.0 (3.9)			
Stevens Creek	Right	1.30 (8.75)	1.02 (8.58)	0.88 (8.19)	0.58 (6.80)	0.30 (4.79)	0.13 (0.48)
Stevens Creek	Left	1.28 (7.33)	1.15 (7.05)	1.45 (6.66)	1.26 (5.49)	0 (0)	0 (0)
Sunnyvale Complex	Left & Right	4.92 (10.44)	4.68 (10.18)	4.22 (9.63)	--	--	--

18.0 Year 50 Without Project Results

The results of the year 50 HEC-RAS analyses found that the water surface elevations did not change significantly from year 0 to year 50. From the coincident frequency analysis (Section 12) it was found that the year 50 sea level change has little affect on the downstream boundary conditions, such that there is little change between year 0 and year 50 water surface elevations. Therefore, there is little to no change in the volume of water leaving the streams and entering the floodplains from year 0 to year 50. As a result there is no change in the fluvial flood inundation maps for year 0 and year 50 and Plates 47 to 73 represent both the year 0 and year 50 riverine floodplains.

19.0 With-Project Riverine Hydraulics Analysis

The scope of the shoreline protection project was reduced subsequent to the without-project hydraulics analysis. As of the time of this writing, only the section of coastline between the Guadalupe River (Alviso Slough) and Coyote Creek was being considered for coastal levee construction. As a result, the with-project hydraulics analysis was limited to those two watercourses.

Proposed coastal levees will tie in to existing riverine levees and have a maximum crest elevation of 16.5 ft (NAVD 88). An aerial view of the study area and subject streams is presented in Figure 19.1. The proposed geometry would not reduce the available flow area or constrict the flow in the channel; therefore, it will not have an effect on water surface elevations in Guadalupe River/Alviso Slough or Coyote Creek.

HEC-RAS models of Coyote Creek and Guadalupe River/Alviso Slough used in the without-project analysis were modified per the proposed design. The left (south) levee crest elevation on Coyote Creek was increased from 15.81 ft to 16.5 ft NAVD 88 at River Station (RS) 74+05. The right (east) levee crest elevation was increased from 15 ft to 16.5 ft at RS 223+29.96 in the Guadalupe River model. No other changes were to the without-project Coyote Creek or Guadalupe River without-project model geometries.

Without-project coincident frequency analyses (described previously in this document) assumed that coastal water surface elevations and riverine flows are independent. Subsequent to the original study, it was shown that flow in the Guadalupe River is well-correlated with storm surge, and that tidal residuals of up to two feet may be expected due to the correlation. The maximum tidewater elevation modeled under without-project conditions was 13 feet. Maximum tidewater elevations were increased in the with-project models to 15 feet to account for storm surge effects. Minimum tidewater elevation in both without- and with-project conditions was 2.83 ft NAVD 88.

Flow hydrographs representing the 1%, 0.4% and 0.2% annual chance exceedance (100-, 250-, and 500-year) events were used for the with-project analysis. Federally constructed riverine levees on both Coyote Creek and Guadalupe River were designed to safely contain the 1% annual chance exceedance (100-year) event. Flows of magnitude equal to or less than the 1% annual chance exceedance (100-year) event will be contained in the channels within the study area.

Modeling results are summarized in Tables 19.1 and 19.2. Neither modification of the cross section geometries (to account for the coastal levee) nor increasing the tidewater elevation to a maximum value of 15 ft NAVD 88 had a significant effect on predicted backwater profiles or breakout flow rates.

TABLE 19.1. COYOTE CREEK BREAKOUT FLOWS

Downstream Boundary Condition (ft NAVD 88)	With or Without Shoreline Levee	Peak Breakout Flow Rate (cfs)		
		1% ACE Event	0.4% ACE Event	0.20% ACE Event
2.83	Without	16	60	91
	With	16	60	91
15	Without	16	60	91
	With	16	60	91
Change Due to Tidewater Elevation (%)		0	0	0
Change Due to Shoreline Levee (%)		0	0	0

TABLE 19.2. GUADALUPE RIVER BREAKOUT FLOWS

Downstream Boundary Condition (ft NAVD 88)	With or Without Shoreline Levee	Peak Breakout Flow Rate (cfs)		
		1% ACE Event	0.4% ACE Event	0.20% ACE Event
2.83	Without	0	1850	3393
	With	0	1850	3393
15	Without	0	1874	3416
	With	0	1874	3416
Change Due to Tidewater Elevation (%)		0	1	1
Change Due to Shoreline Levee (%)		0	0	0

A cross section plot of Coyote Creek at the location of the proposed coastal levee is presented in Figure 19.1. With- and without-project water surface elevations are plotted. There is no change in the predicted water surface elevation due to the proposed coastal levee. Four 1% annual chance exceedance scenarios are presented in a longitudinal profile plot of Coyote Creek in Figure 19.2: without-project, 2.83 feet downstream boundary condition (DSBC); with-project, 2.83 feet DSBC; without-project, 15 feet DSBC; and, with-project 15 feet DSBC. With- and without-project profiles are identical for the same DSBC. The higher downstream boundary condition results in a higher backwater profile, with the effects tapering off until Highway 237, beyond which there are no effects on the backwater profile due to downstream boundary condition.

A cross section plot of Guadalupe River/Alviso Slough at the location of the proposed coastal levee is presented in Figure 19.3. With- and without-project water surface elevations are plotted. There is no change in the predicted water surface elevation due to the proposed coastal levee. Four 1% annual chance exceedance scenarios are presented in a longitudinal profile plot of Guadalupe River/Alviso Slough in Figure 19.4: without-project, 2.83 feet downstream boundary condition (DSBC); with-project, 2.83 feet DSBC; without-project, 15 feet DSBC; and, with-project 15 feet DSBC. With- and without-project profiles are identical for the same DSBC. The higher downstream boundary condition results in a higher backwater profile, with the effects tapering off until Highway 237, beyond which there are no effects on the backwater profile due to downstream boundary condition.

As with the without-project analysis described previously in this document, the breakout flow hydrographs were calculated using an unsteady HEC-RAS simulation. The breakout hydrographs were then used as inflow hydrographs for a two-dimensional floodplain hydraulics simulation using FLO-2D. The inflow hydrographs for the two-dimensional models are the same as the without-project condition. Interior drainage capacity for the with-project condition will be identical for the without project condition (Jimenez, personal communication, 2013). Therefore, the floodplain inundation maps are unchanged for the with-project condition. A 0.2% annual chance exceedance floodplain inundation map of Coyote Creek is presented in Plate 50. 0.2% annual chance exceedance floodplain inundation maps of the Guadalupe River are presented in Plates 54 and 55.

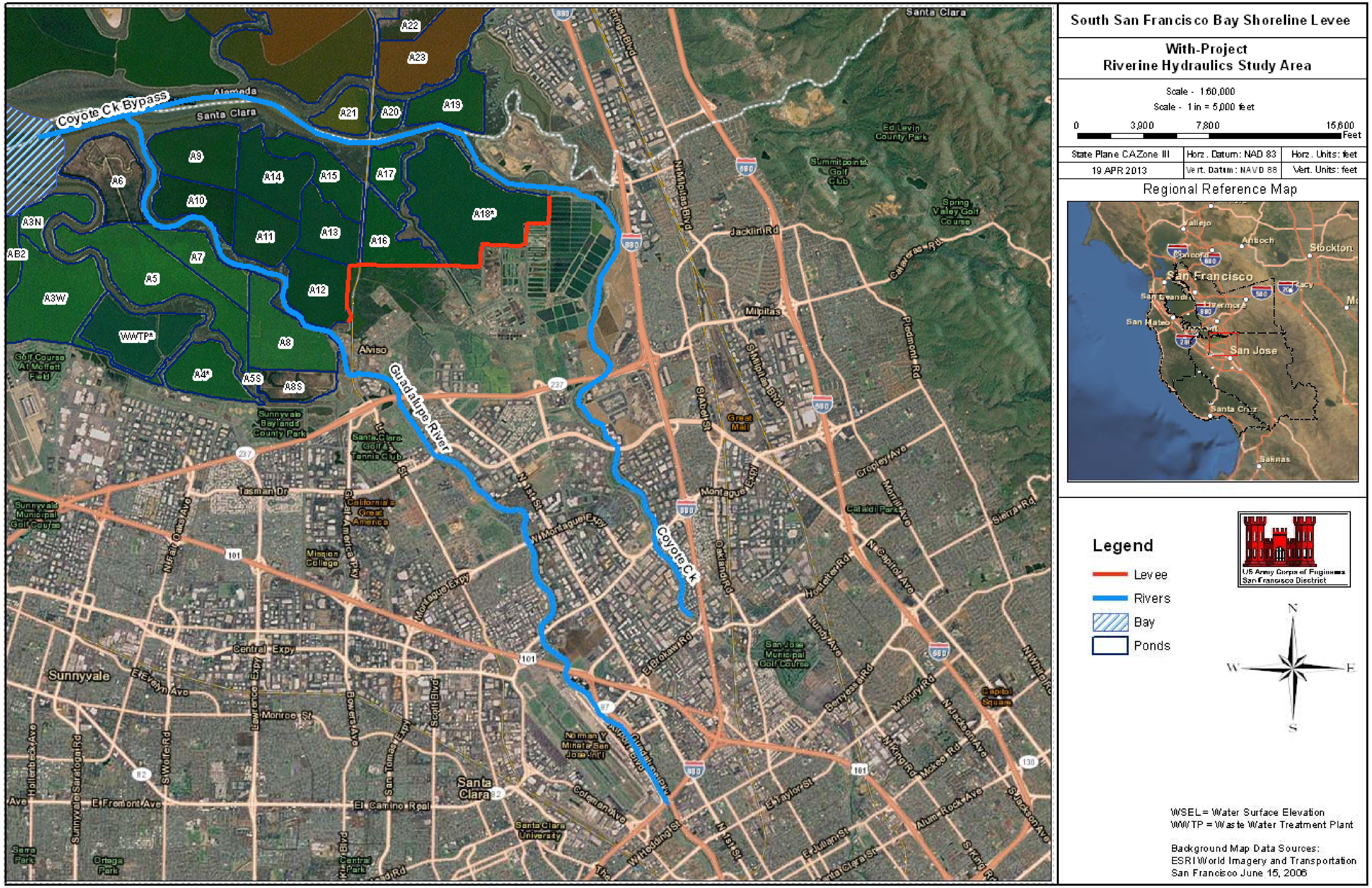

FIGURE 19.1. AERIAL VIEW WITH-PROJECT CONDITIONS RIVERINE HYDRAULICS STUDY AREA

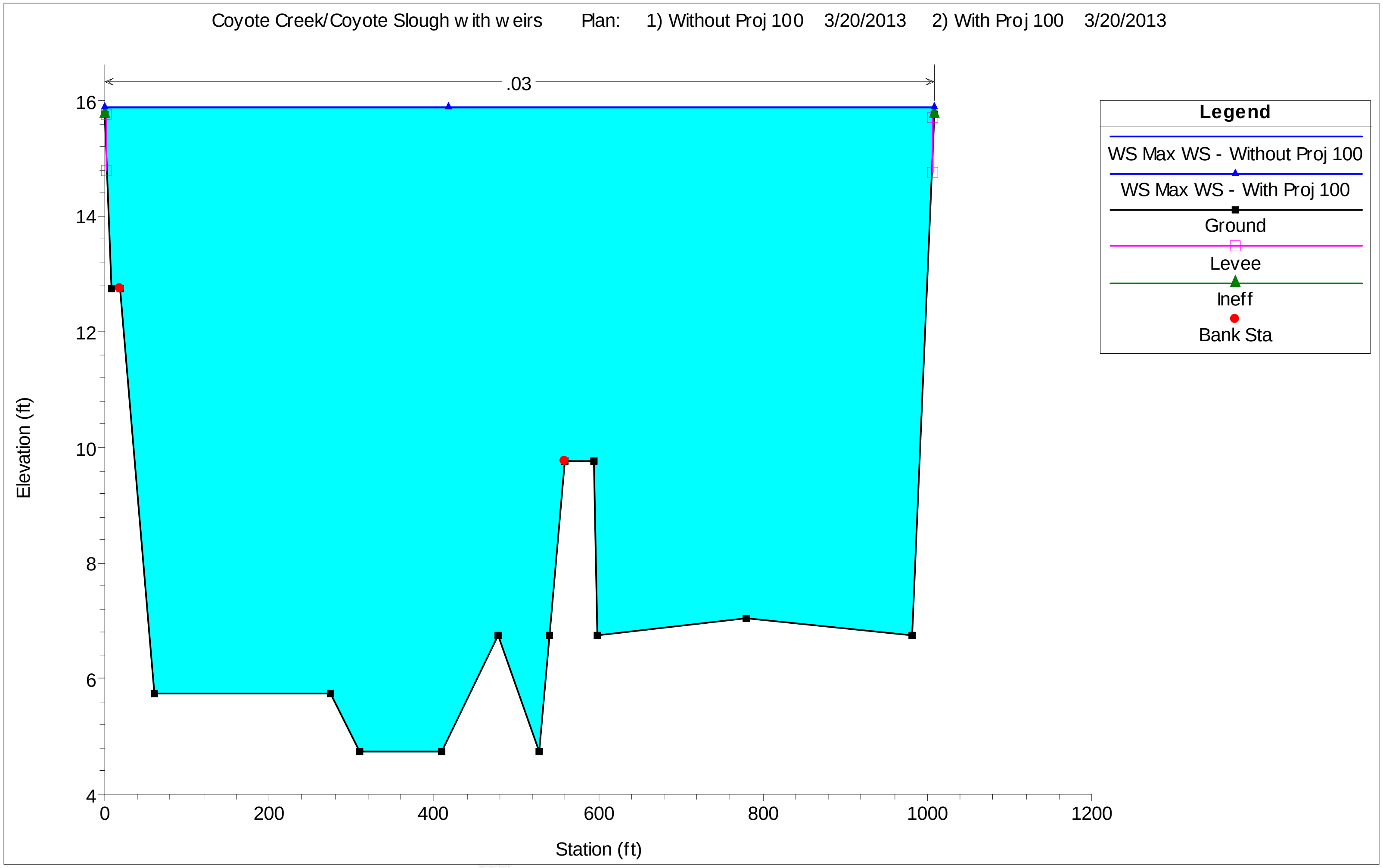

FIGURE 19.2. CROSS SECTION LOCATED CLOSEST TO THE POINT WHERE SHORELINE LEVEE INTERSECTS COYOTE CREEK (STA. 74+05)

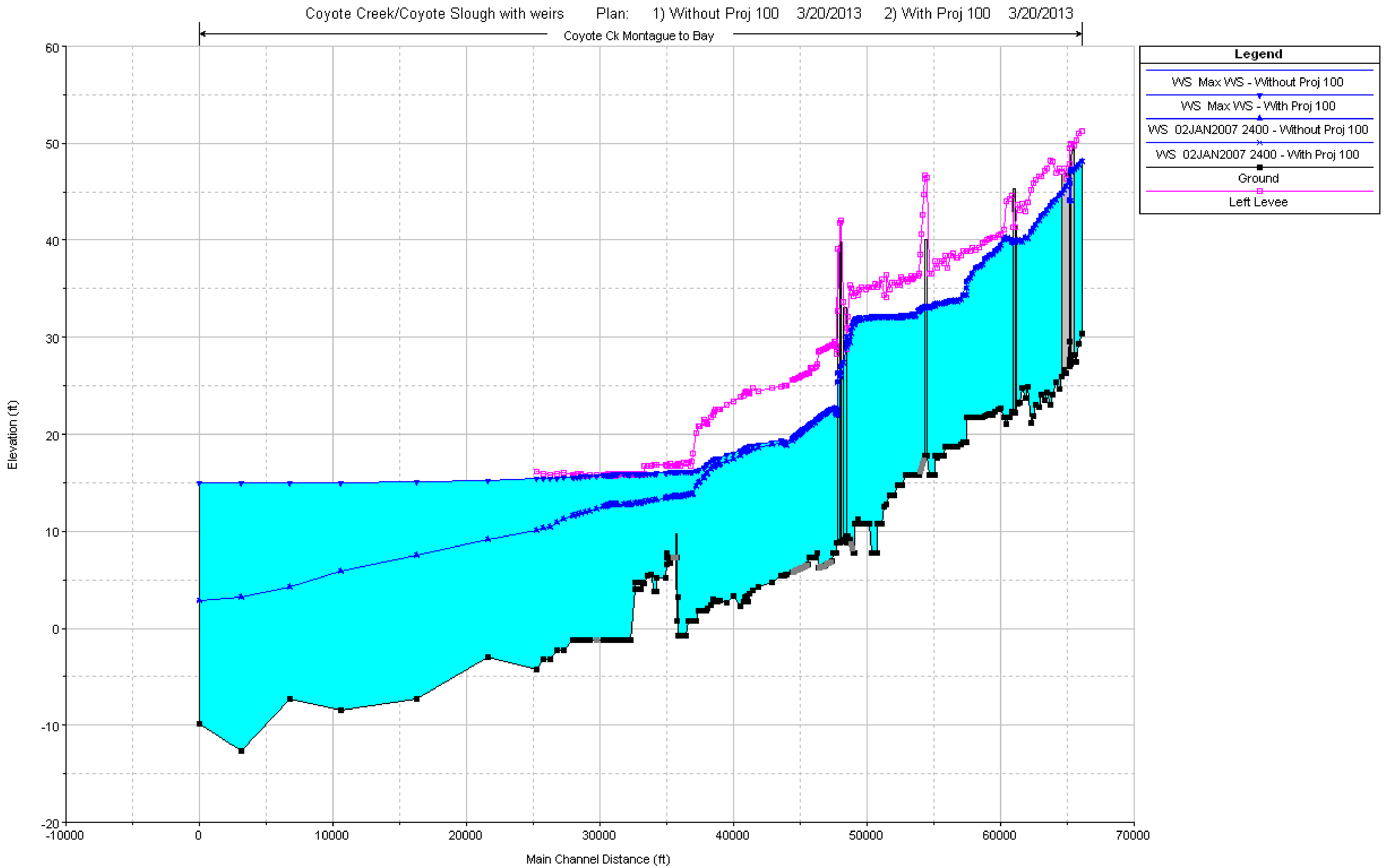

FIGURE 19.3. LONGITUDINAL PROFILE OF COYOTE CREEK SHOWING 1% ACE WATER SURFACES FOR FOUR SCENARIOS: (1) WITHOUT-PROJECT GEOMETRY, 2.83 FT DOWNSTREAM BOUNDARY CONDITION; (2) WITH-PROJECT GEOMETRY, 2.83 FT DOWNSTREAM BOUNDARY CONDITION; (3) WITHOUT-PROJECT GEOMETRY, 15 FT DOWNSTREAM BOUNDARY CONDITION; AND (4) WITH-PROJECT GEOMETRY, 15 FT DOWNSTREAM BOUNDARY CONDITION. THE WITH- AND WITHOUT-PROJECT BACKWATER PROFILES ARE IDENTICAL.

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FIGURE 19.4. CROSS SECTION LOCATED CLOSEST TO THE POINT WHERE THE PROPOSED SHORELINE LEVEE INTERSECTS GUADALUPE RIVER (STA 223+29.96)

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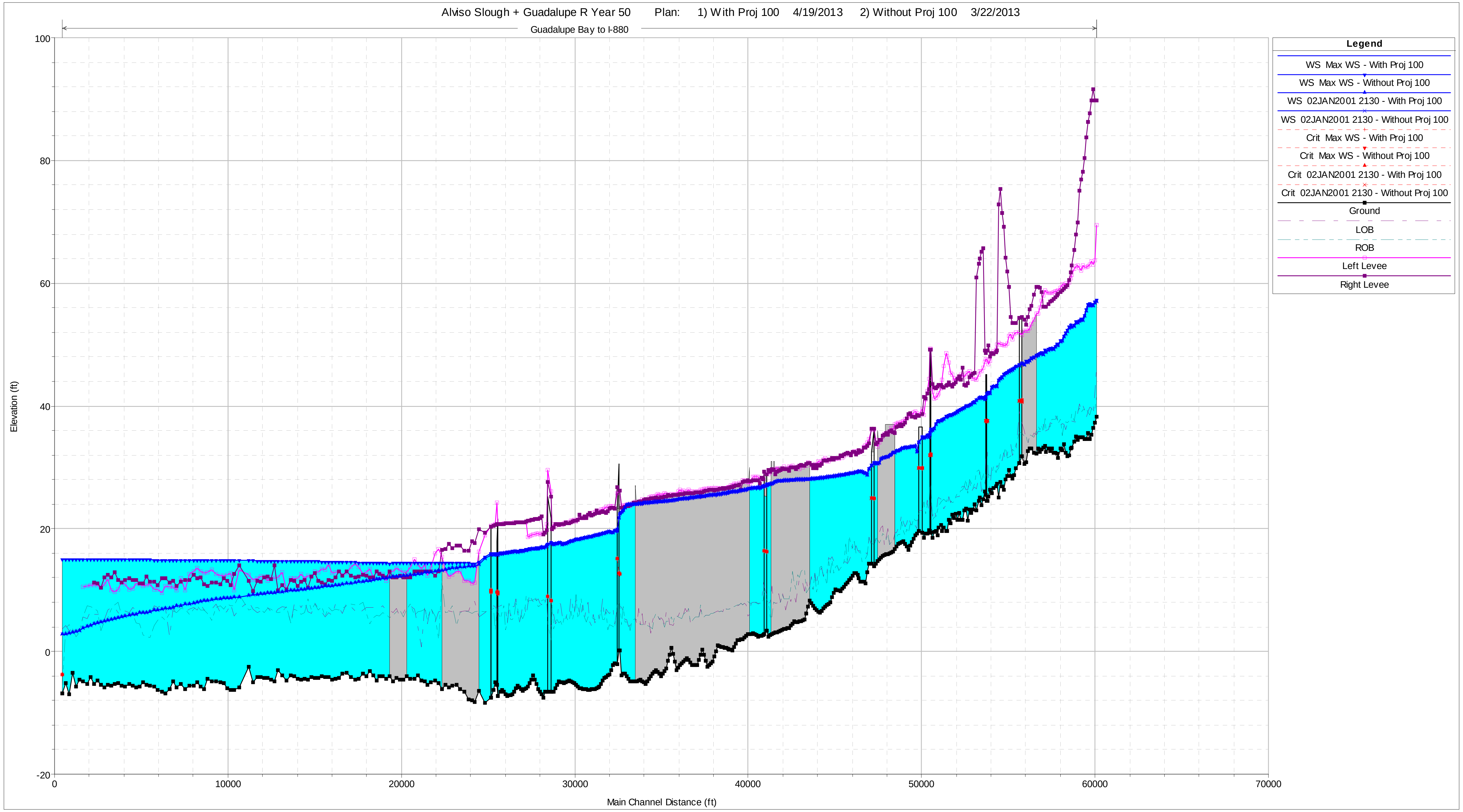

FIGURE 19.5. LONGITUDINAL PROFILE OF GUADALUPE RIVER SHOWING 1% ACE WATER SURFACES FOR FOUR SCENARIOS: (1) WITHOUT-PROJECT GEOMETRY, 2.83 FT DOWNSTREAM BOUNDARY CONDITION; (2) WITH-PROJECT GEOMETRY, 2.83 FT DOWNSTREAM BOUNDARY CONDITION; (3) WITHOUT-PROJECT GEOMETRY, 15 FT DOWNSTREAM BOUNDARY CONDITION; AND (4) WITH-PROJECT GEOMETRY, 15 FT DOWNSTREAM BOUNDARY CONDITION. THE WITH- AND WITHOUT-PROJECT BACKWATER PROFILES ARE IDENTICAL.

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19.0 References

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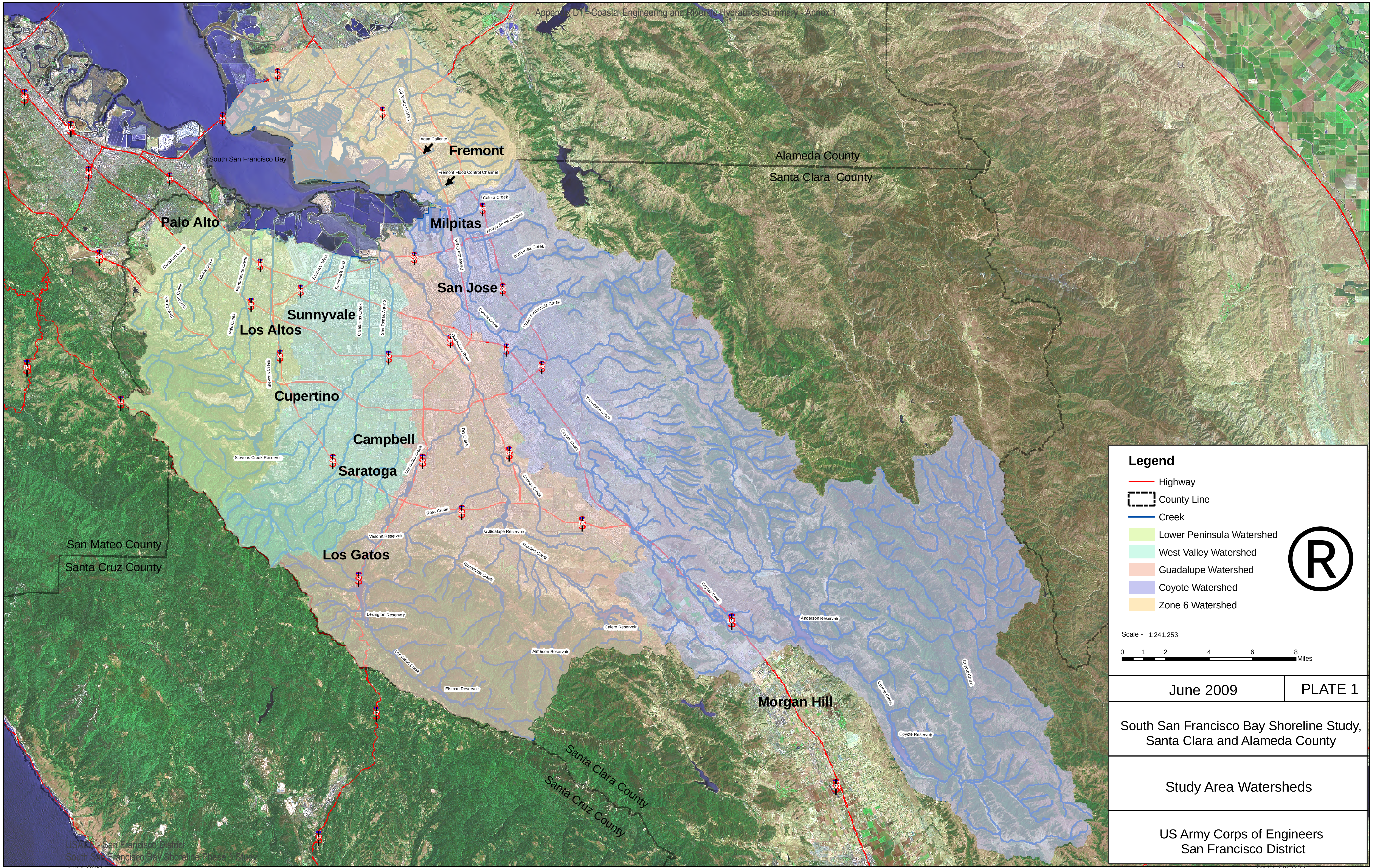
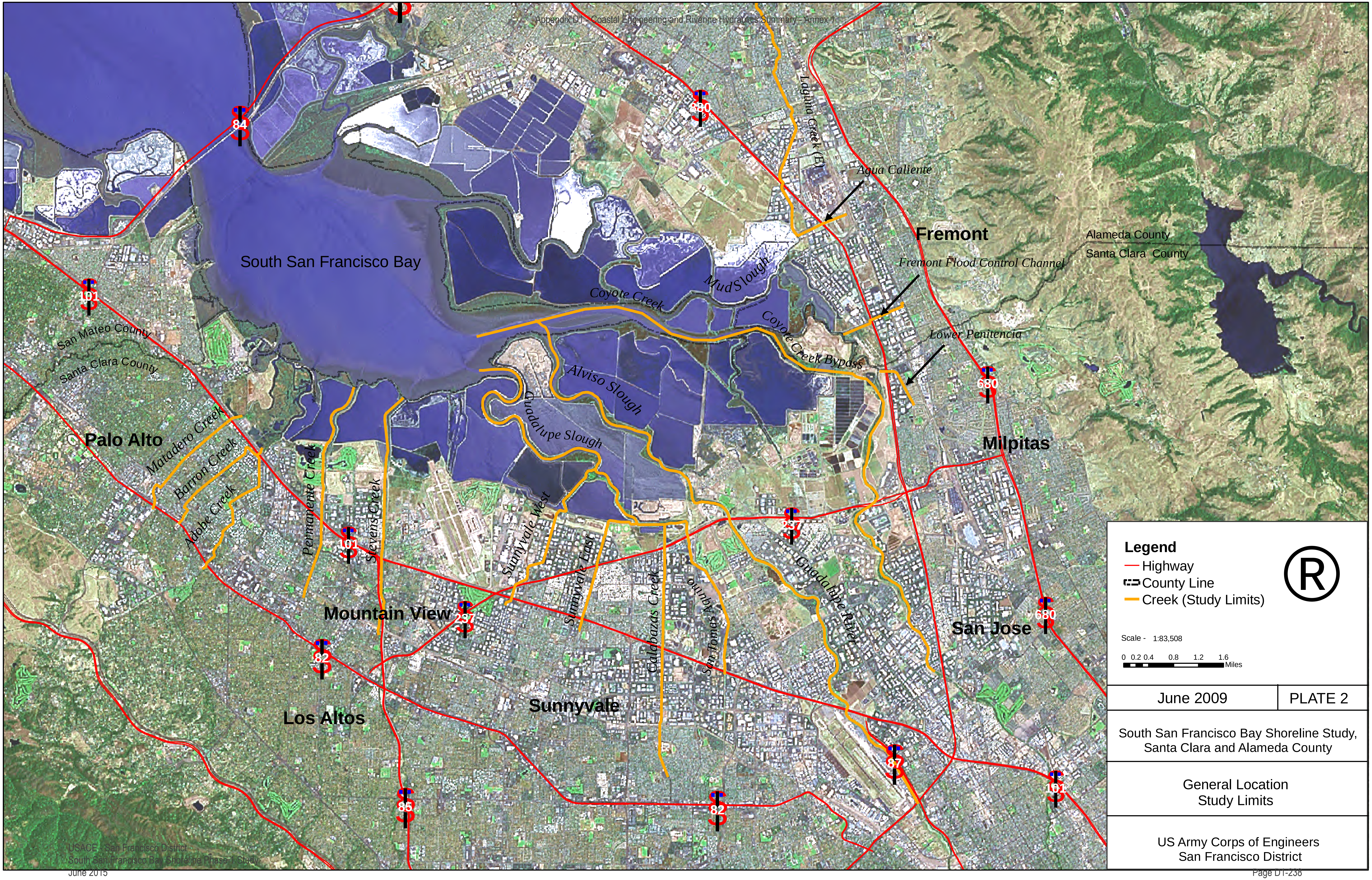


South San Francisco Bay
Fremont
Alameda County
Santa Clara County
Palo Alto
Milpitas
San Jose
Sunnyvale
Los Altos
Cupertino
Campbell
Saratoga
Los Gatos
San Mateo County
Santa Cruz County
Morgan Hill
Santa Clara County
Santa Cruz County
Stevens Creek Reservoir
Vasona Reservoir
Guadalupe Reservoir
Lexington Reservoir
Calero Reservoir
Almaden Reservoir
Elsman Reservoir
Anderson Reservoir
Coyote Reservoir
Legend
Highway
County Line
Creek
Lower Peninsula Watershed
West Valley Watershed
Guadalupe Watershed
Coyote Watershed
Zone 6 Watershed
Scale - 1:241,253
0 1 2 4 6 8 Miles
June 2009
PLATE 1
South San Francisco Bay Shoreline Study, Santa Clara and Alameda County
Study Area Watersheds
US Army Corps of Engineers San Francisco District


South San Francisco Bay
Fremont
Milpitas
San Jose
Palo Alto
Mountain View
Los Altos
Sunnyvale
Alameda County
Santa Clara County
San Mateo County
Santa Clara County
Laguna Creek (E)
Agua Caliente
Fremont Flood Control Channel
Lower Penitencia
Coyote Creek
Mud Slough
Coyote Creek Bypass
Alviso Slough
Guadalupe Slough
Matadero Creek
Barron Creek
Adobe Creek
Permanente Creek
Stevens Creek
Sunnyvale West
Sunnyvale East
Calabazas Creek
San Tomas Aquino
Guadalupe River
84
880
101
680
237
85
82
87
Legend
Highway
County Line
Creek (Study Limits)
Scale - 1:83,508
0 0.2 0.4 0.8 1.2 1.6 Miles
June 2009
PLATE 2
South San Francisco Bay Shoreline Study, Santa Clara and Alameda County
General Location Study Limits
US Army Corps of Engineers San Francisco District

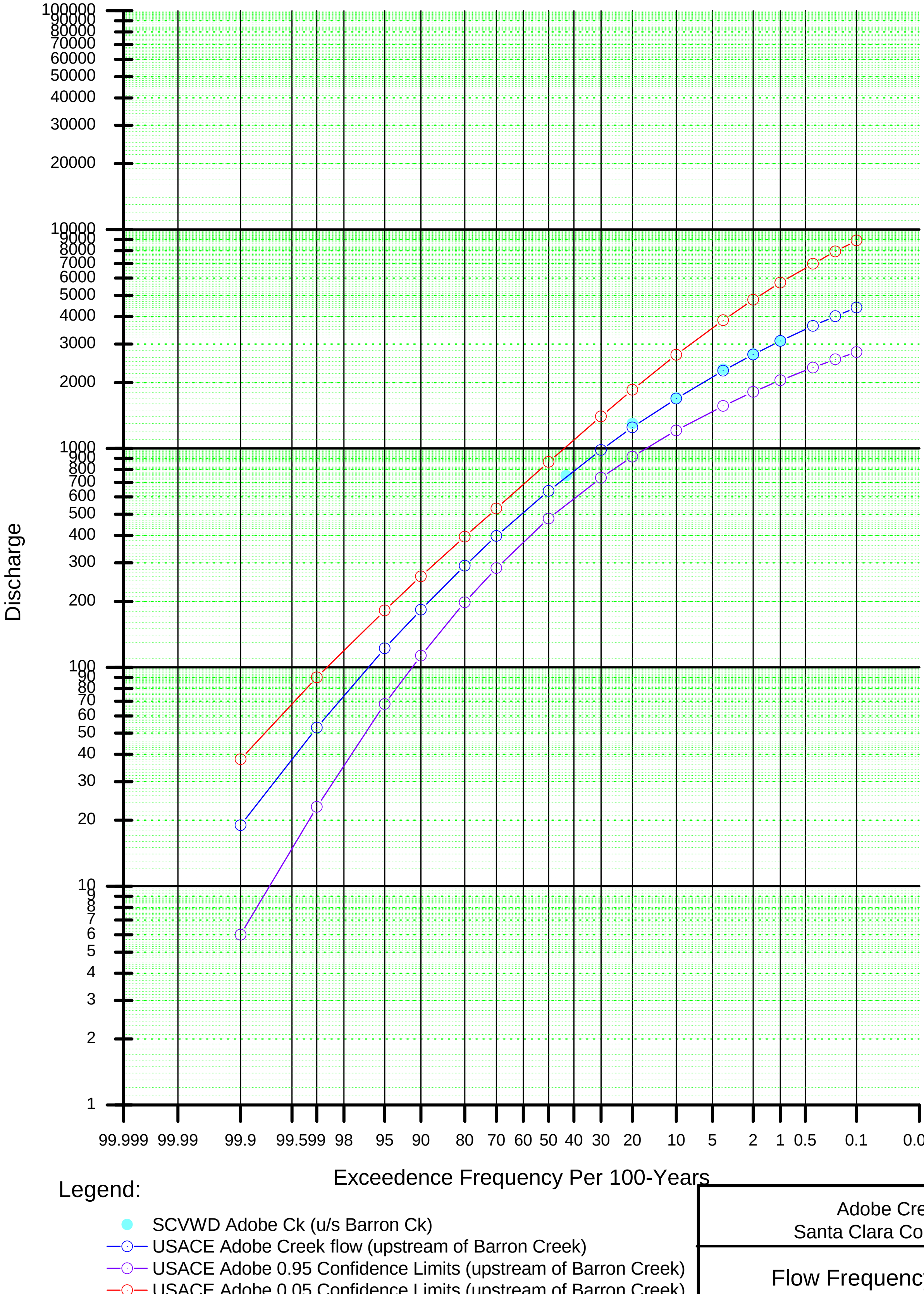


Plate 3

Adobe Creek,
Santa Clara County, CA

Flow Frequency Curve
Adobe Creek
upstream of Barron Creek

US Army Corps of Engineers
San Francisco District

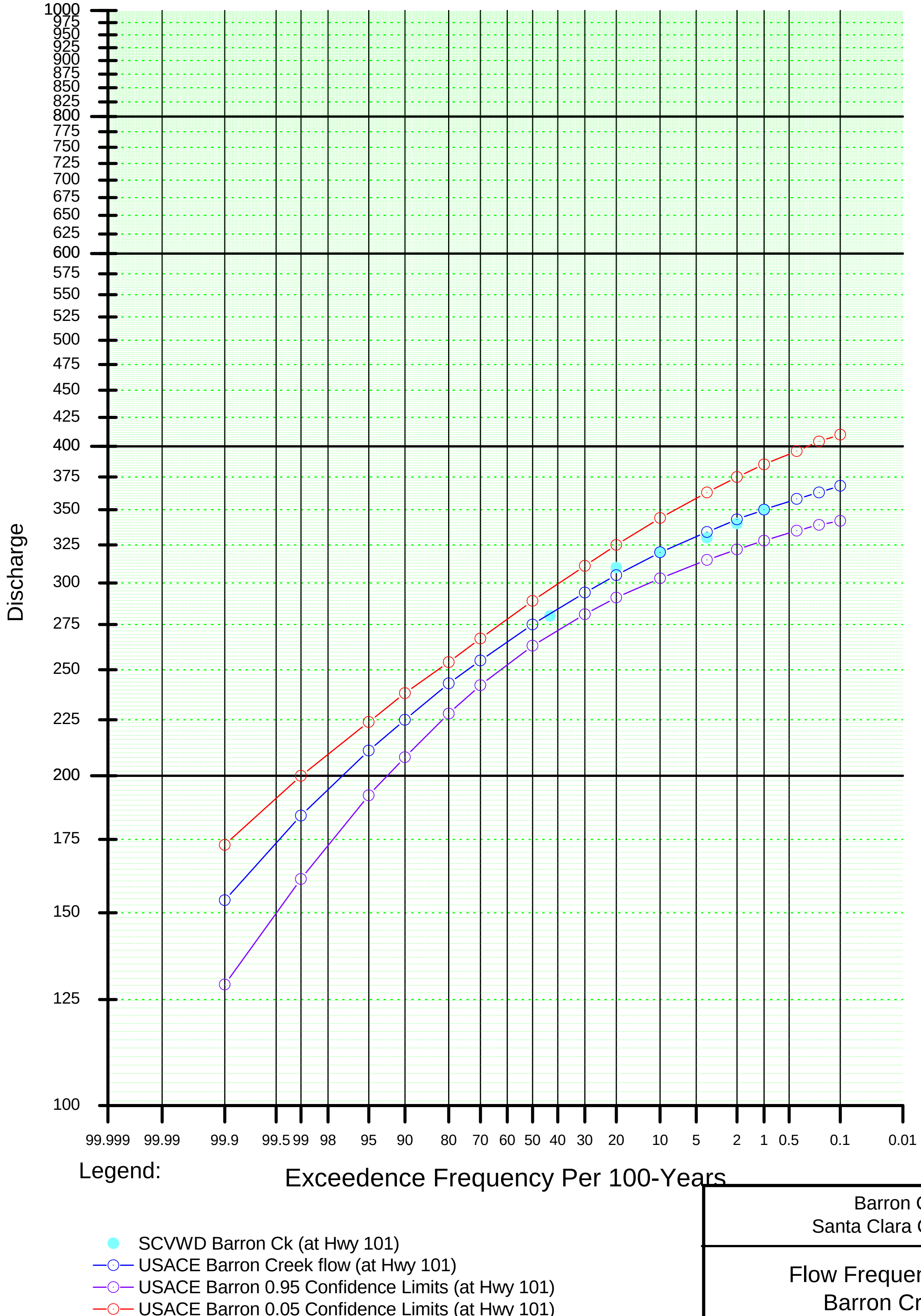


Plate 4

Barron Creek,
Santa Clara County, CA

Flow Frequency Curve
Barron Creek at
Highway 101

US Army Corps of Engineers
San Francisco District

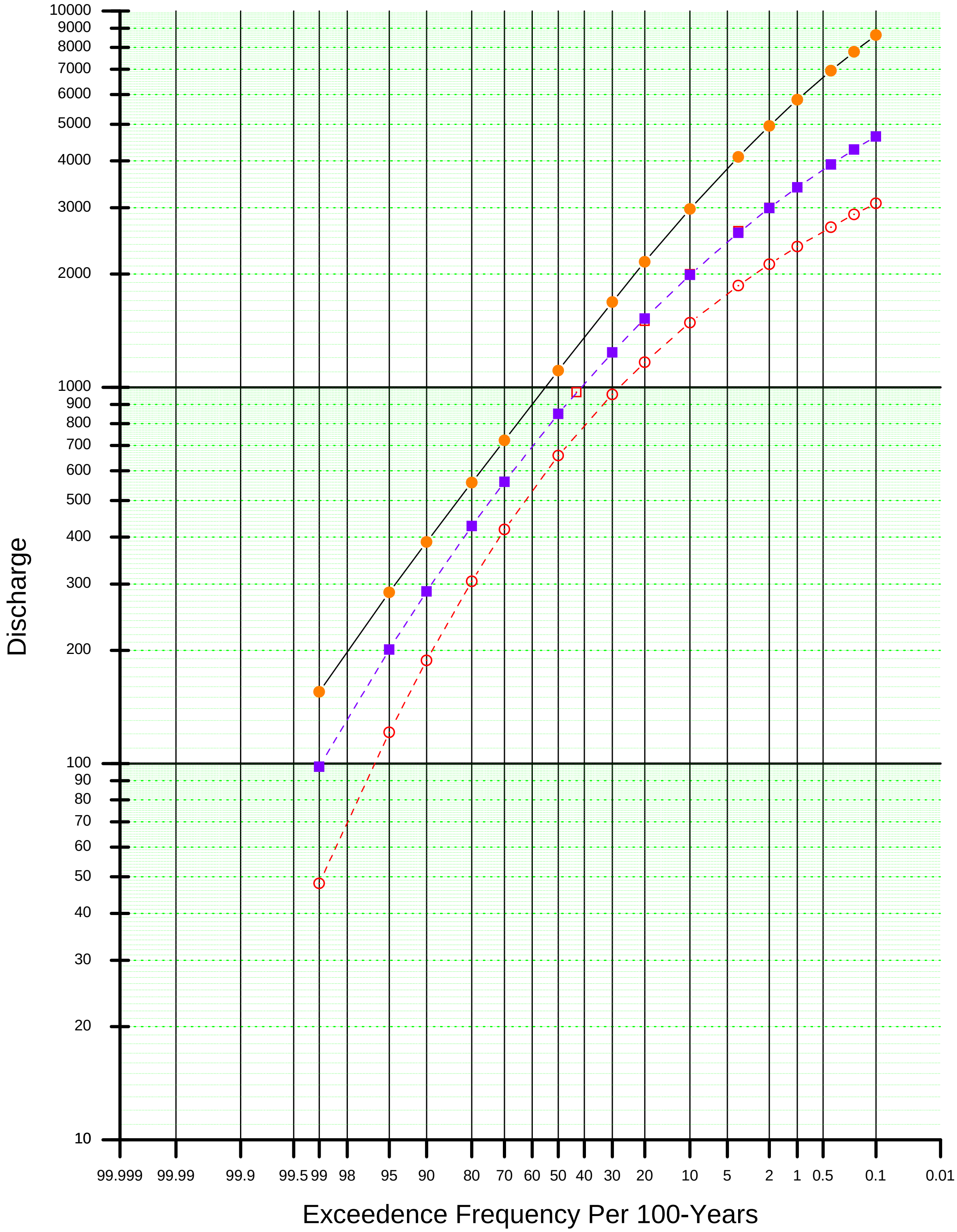


Plate 5

Legend:

SCVWD Adobe - Barron Ck (at Hwy 101)
USACE Adobe-Barron flow (ds of Hwy 101)

Adobe - Barron Creeks,
Santa Clara County, CA

Flow Frequency Curve
Confluence of
Adobe-Barron Cks

US Army Corps of Engineers
San Francisco District

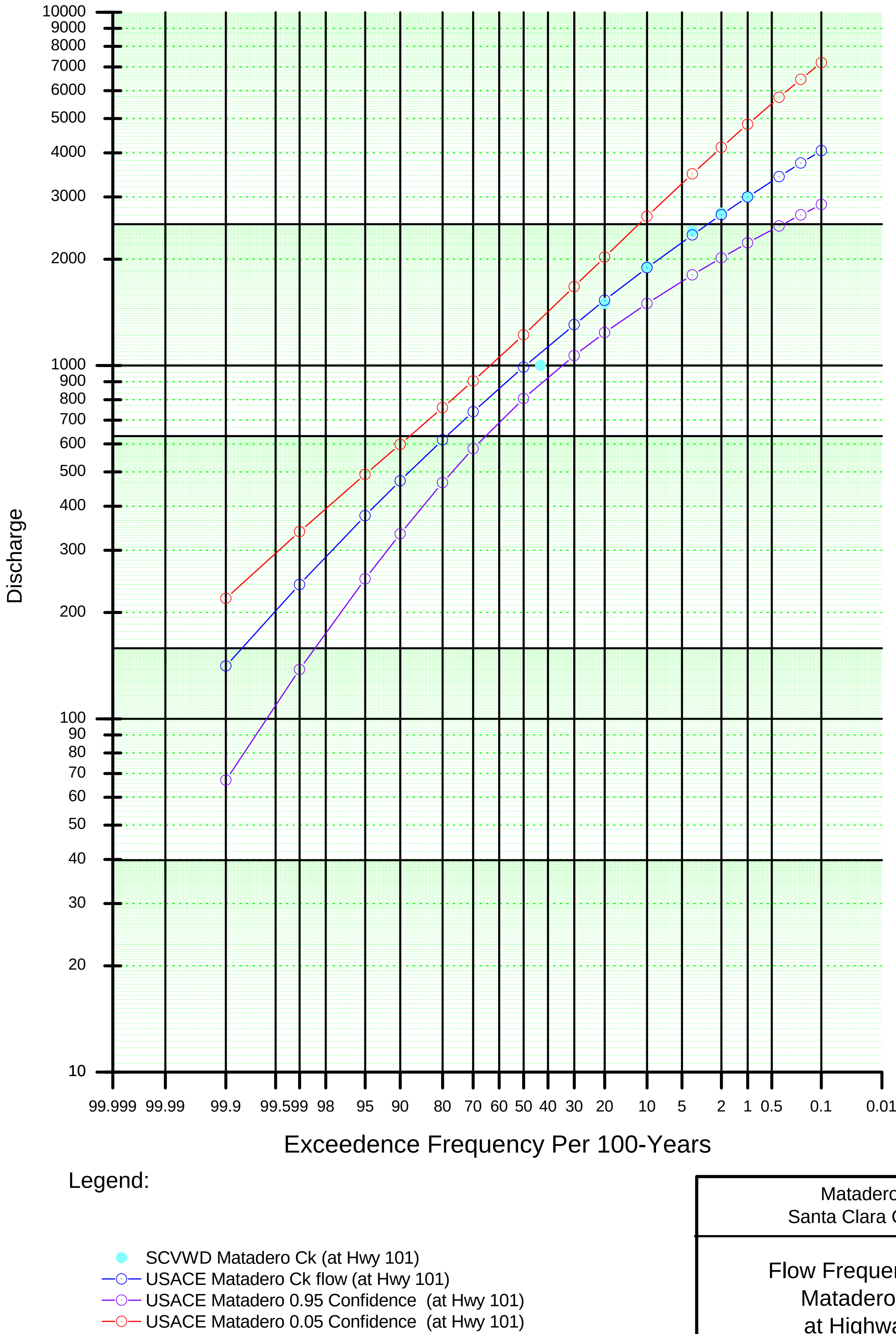


Plate 6

Matadero Creek
Santa Clara County, CA

Flow Frequency Curve
Matadero Creek
at Highway 101

US Army Corps of Engineers
San Francisco District

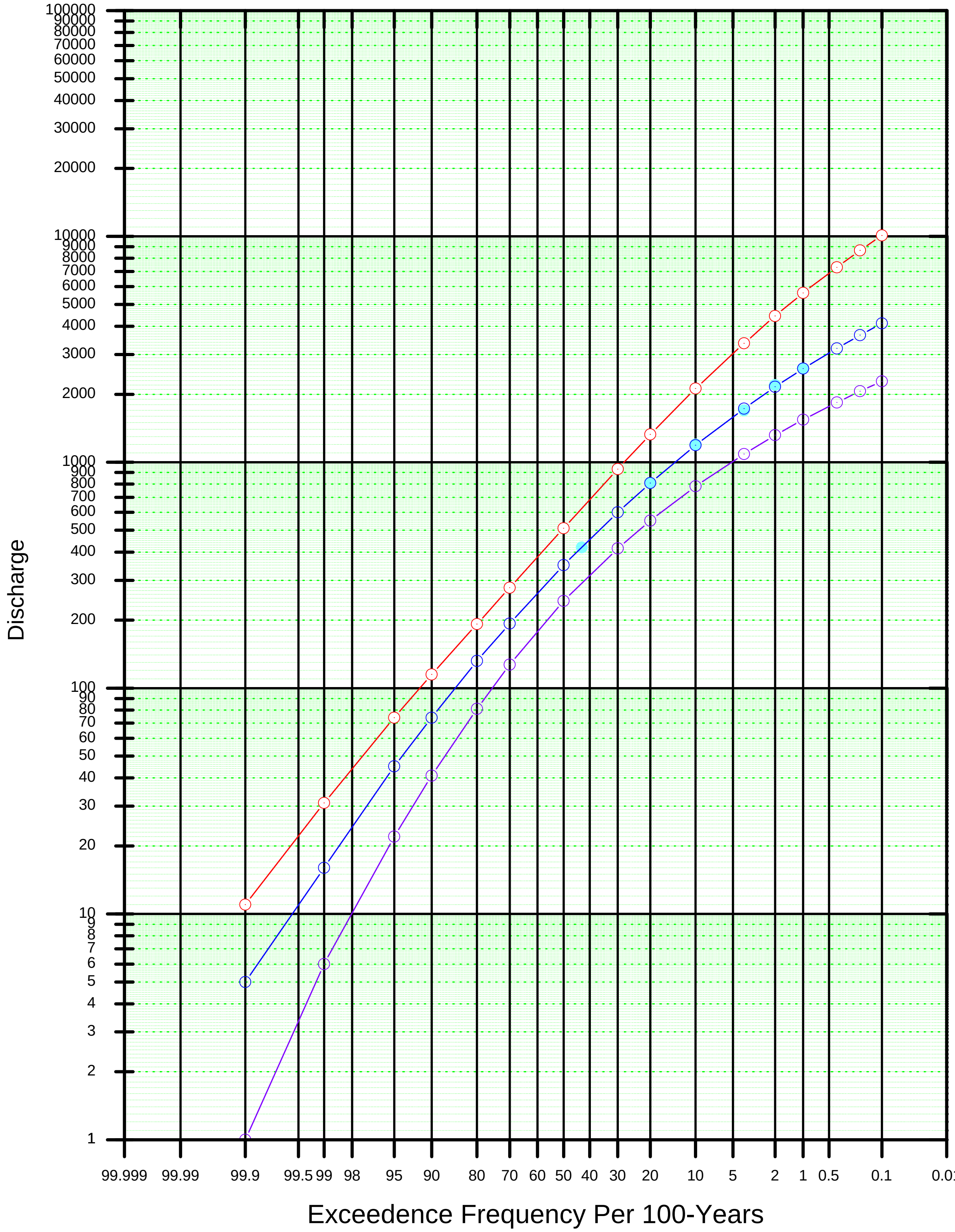


Plate 7

Legend:

- SCVWD Permanente Ck (at Hwy 101)
- USACE Permanente Ck flow (at Hwy 101)
- USACE Permanente 0.95 Confidence Limits (at Hwy 101)
- USACE Permanente 0.05 Confidence Limits (at Hwy 101)

Permanente Creek, Santa Clara County, CA
Flow Frequency Curve Permanente Creek at Highway 101
US Army Corps of Engineers San Francisco District

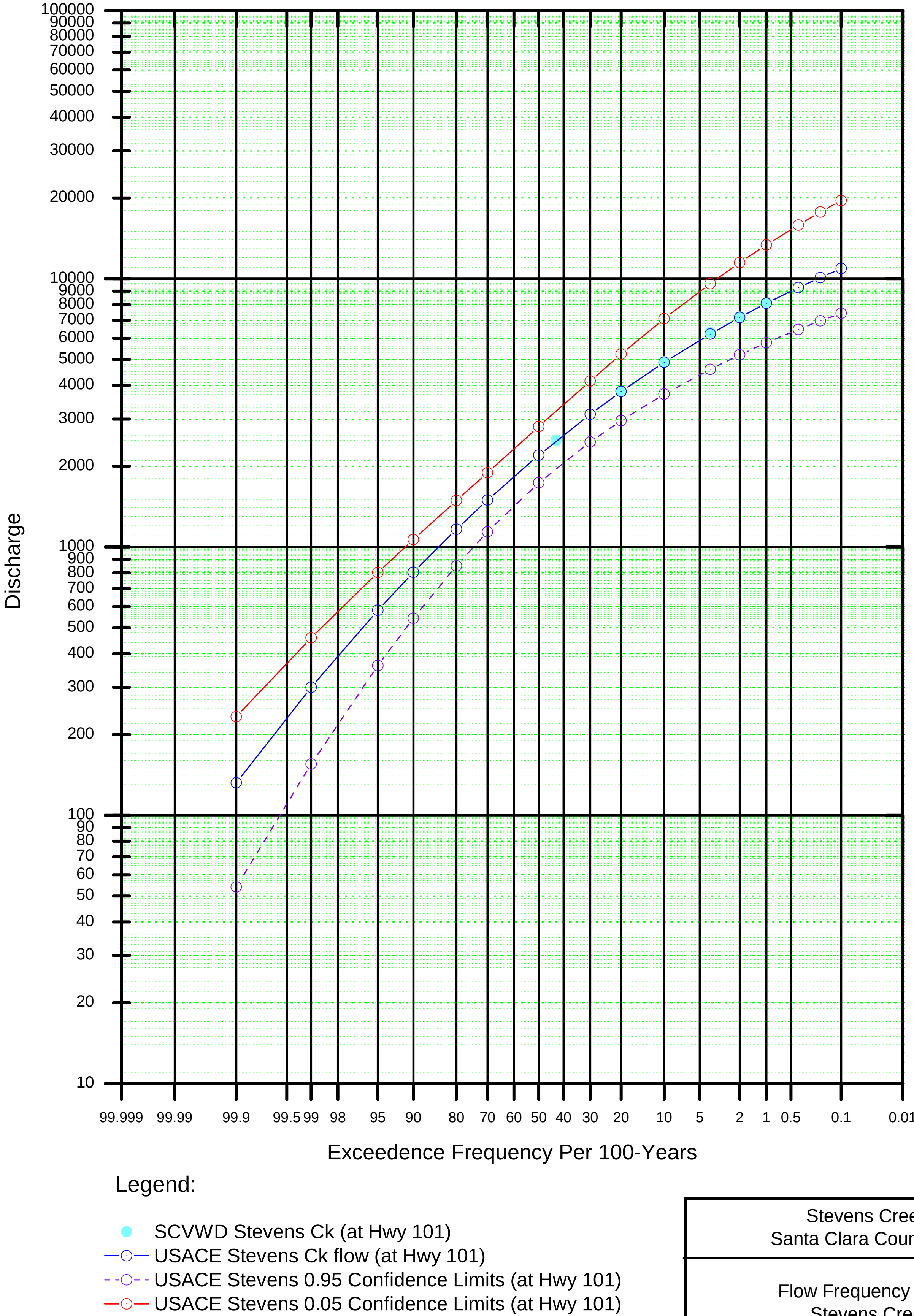


Plate 8

Stevens Creek, Santa Clara County, CA
Flow Frequency Curve Stevens Creek at Highway 101
US Army Corps of Engineers San Francisco District

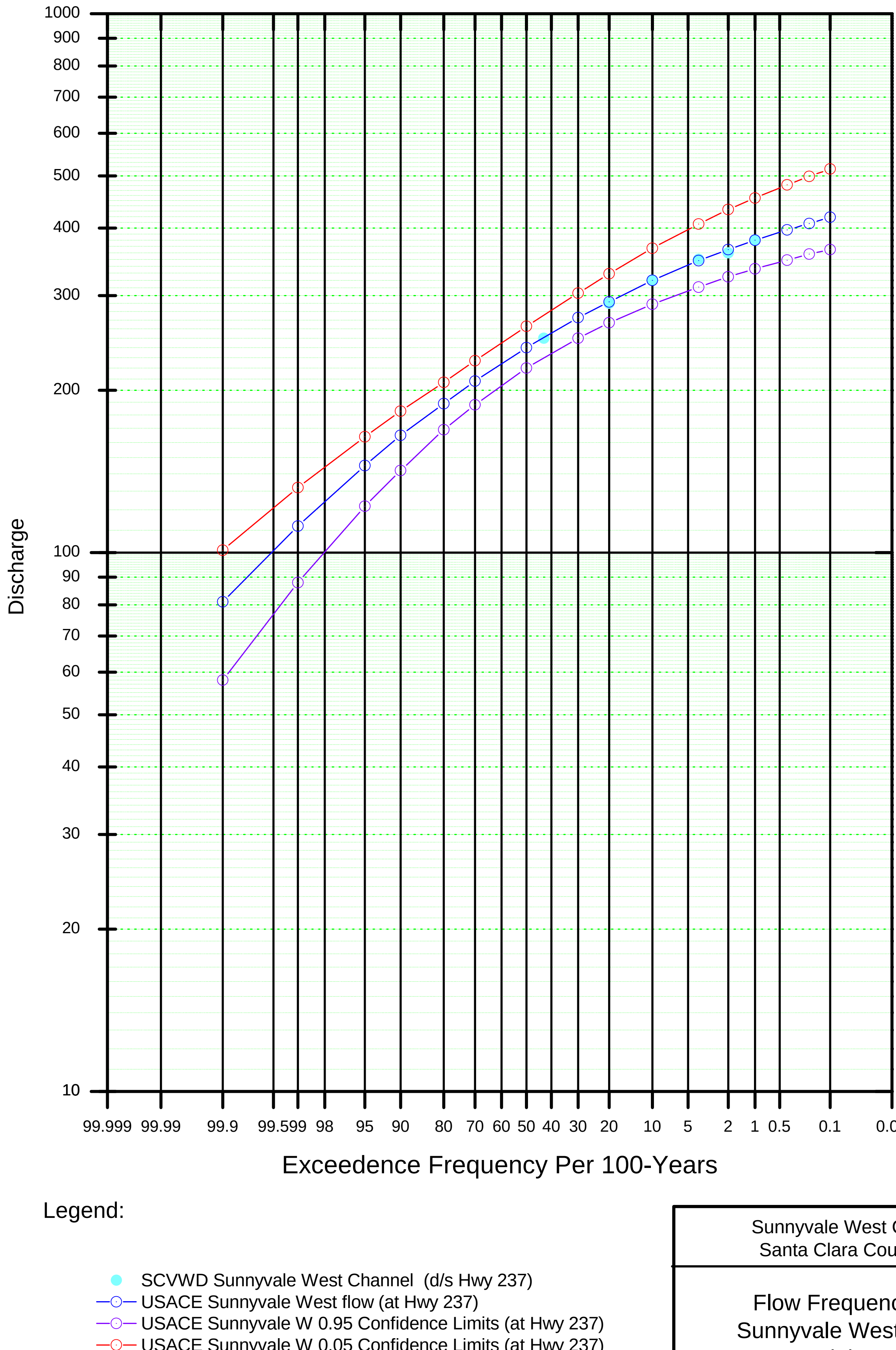


Plate 9

Sunnyvale West Channel, Santa Clara County, CA
Flow Frequency Curve Sunnyvale West Channel at Highway 237
US Army Corps of Engineers San Francisco District

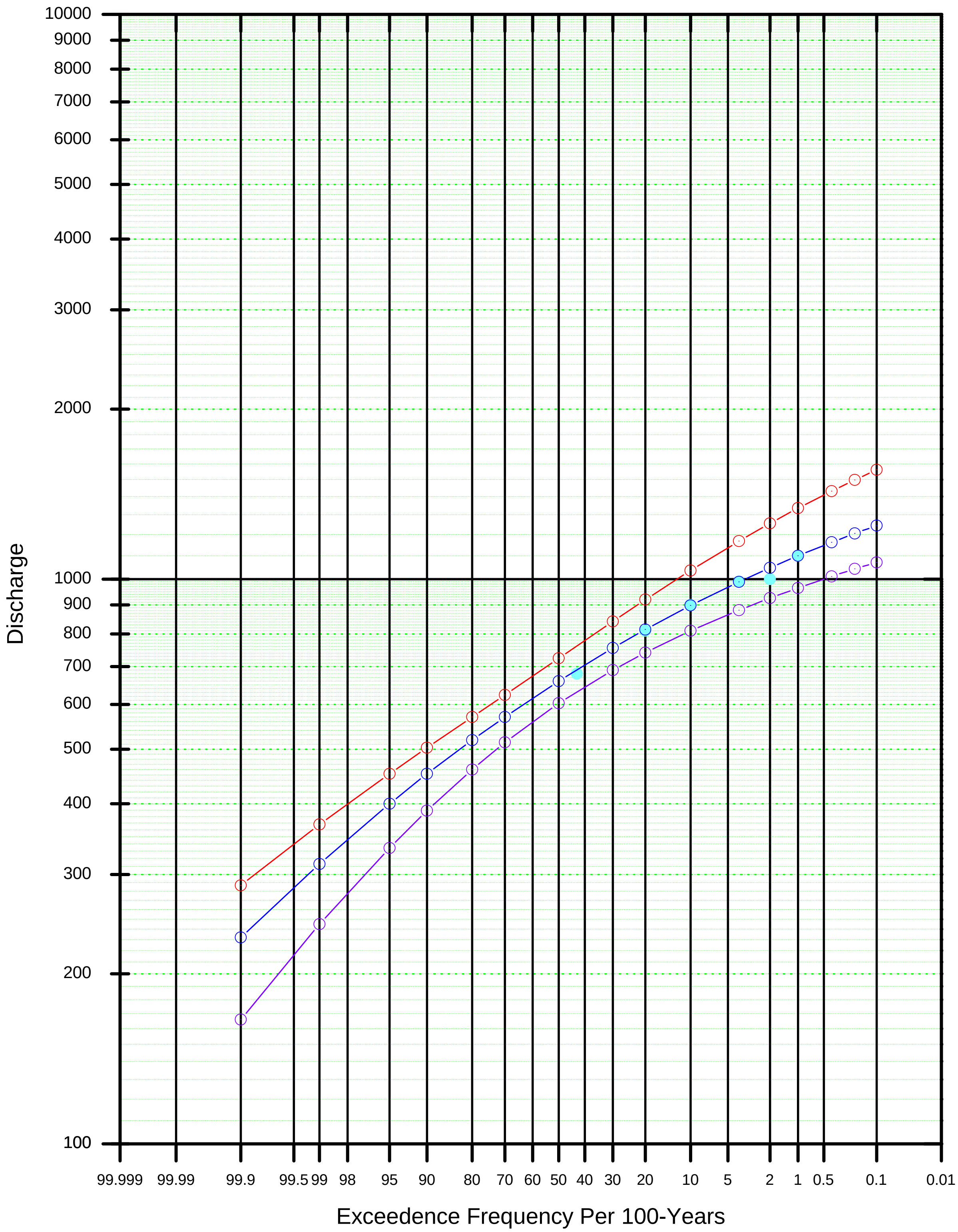


Plate 10

Legend:

- SCVWD Sunnyvale East Channel (d/s Caribbean Drive)
- USACE Sunnyvale East flow (at Caribbean)
- USACE Sunnyvale E. 0.95 Confidence Limits (at Caribbean)
- USACE Sunnyvale E. 0.05 Confidence Limits (at Caribbean)

Sunnyvale East Channel, Santa Clara County, CA
Flow Frequency Curve Sunnyvale East Channel at Carribean St
US Army Corps of Engineers San Francisco District

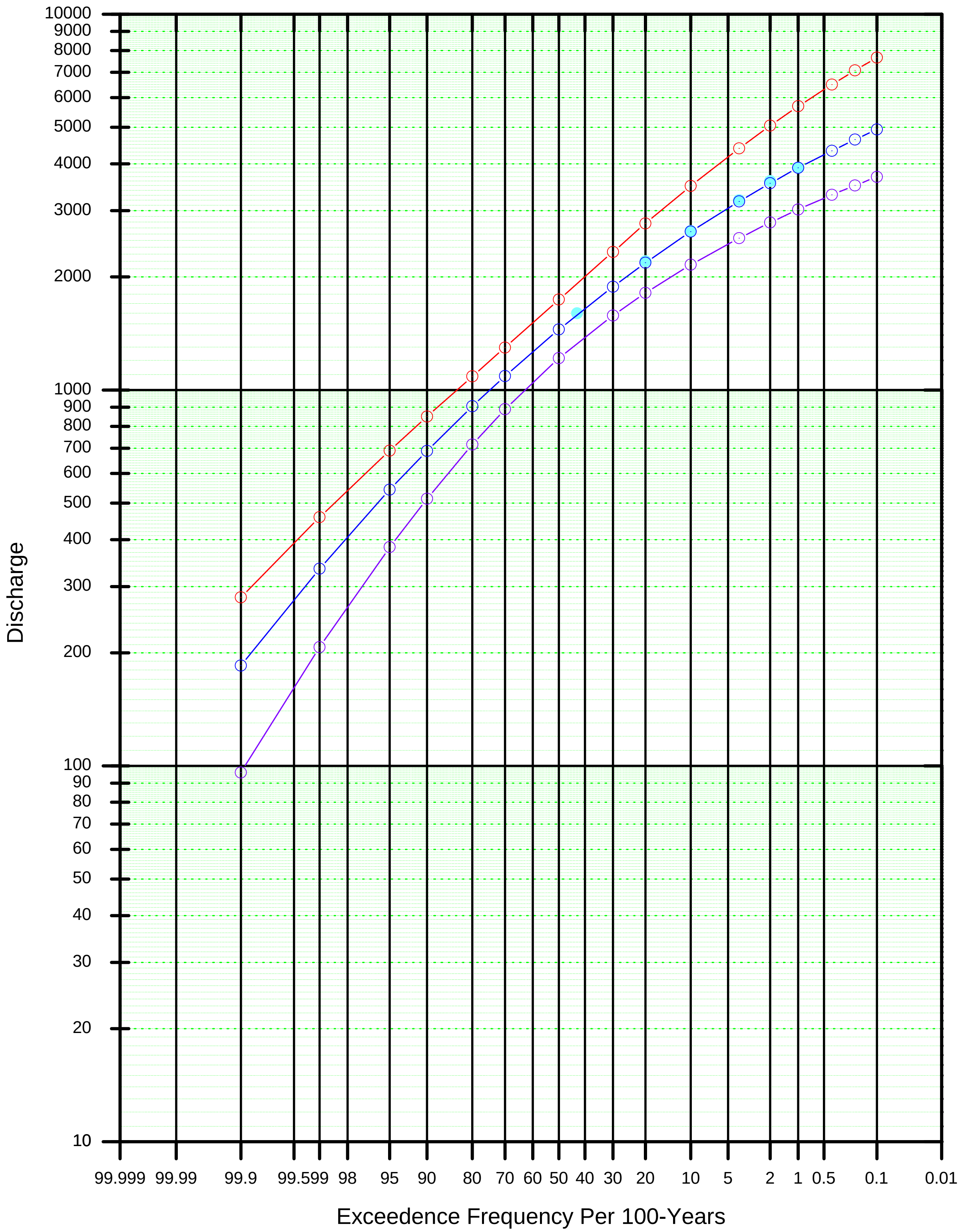


Plate 11

Legend:

- SCVWD Calabazas Ck (d/s Hwy 237)
- USACE Calabazas Crk flow (at Hwy 237)
- USACE Calabazas 0.95 Confidence Limits (at Hwy 237)
- USACE Calabazas 0.05 Confidence Limits (at Hwy 237)

Calabazas Creek, Santa Clara County, CA
Flow Frequency Curve Calabazas Creek at Highway 237
US Army Corps of Engineers San Francisco District

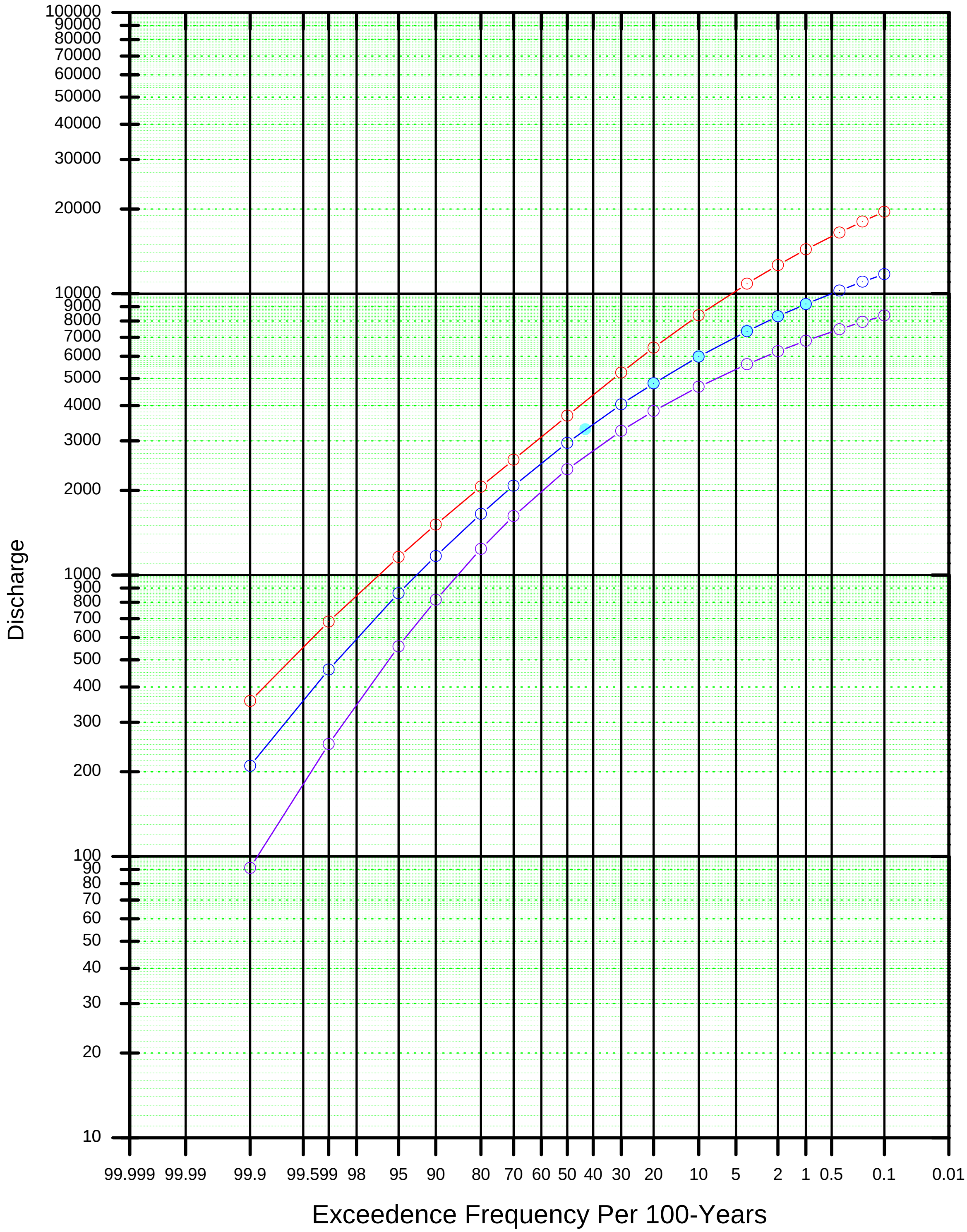


Plate 12

Legend:

- SCVWD San Tomas Aquino (at Hwy 237)
- USACE San Tomas Aquino flow (at Hwy 237)
- USACE San Tomas Aquino 0.95 Confidence Limits (at Hwy 237)
- USACE San Tomas Aquino 0.05 Confidence Limits (at Hwy 237)

San Tomas Aquino Creek,
Santa Clara County, CA

Flow Frequency Curve
San Tomas Aquino Creek
at Highway 237

US Army Corps of Engineers
San Francisco District

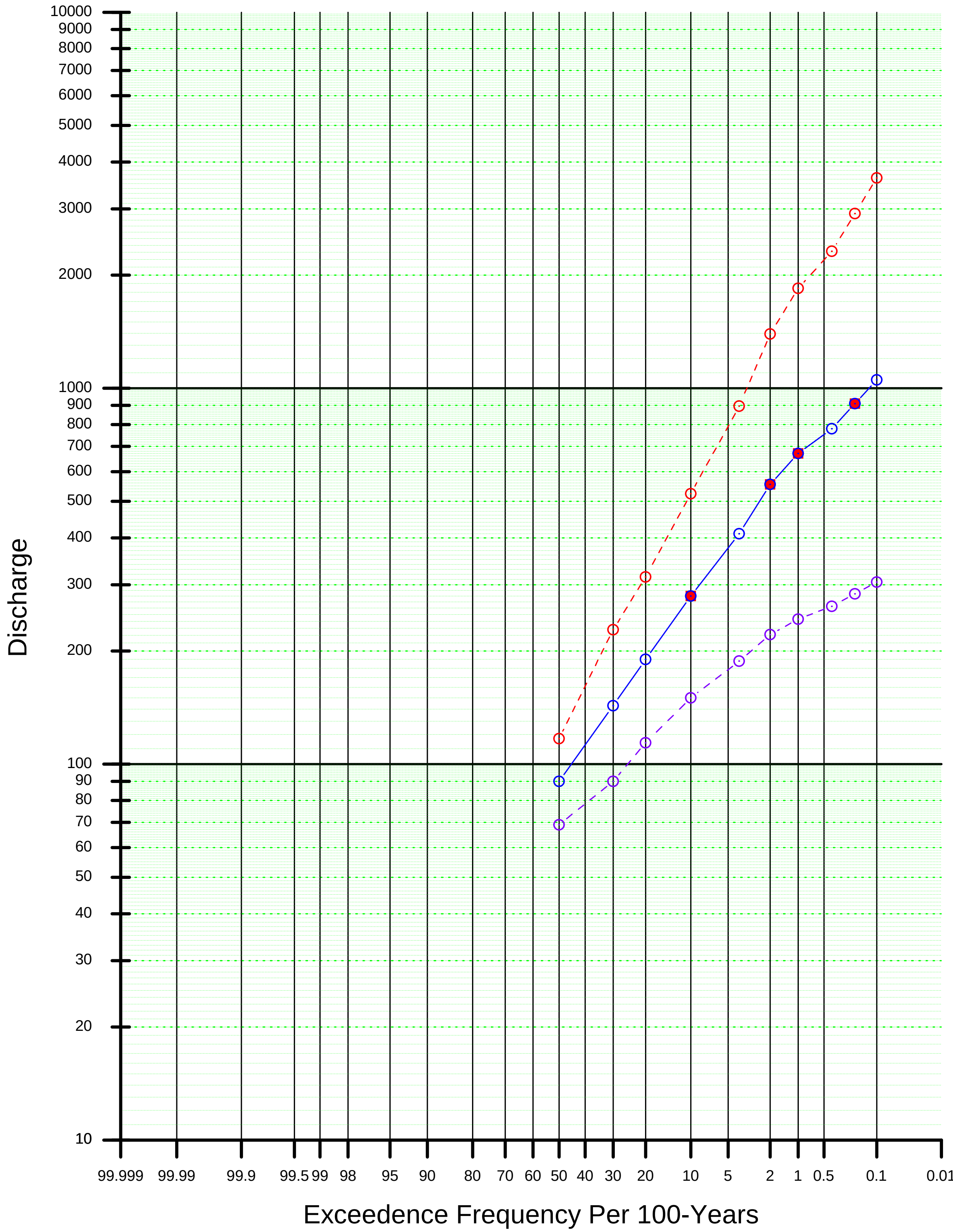


Plate 13

Legend:

- Fremont Flood Control Channel (at Hwy 880)
- USACE Fremont FCC (line B) flow (at Hwy 880)
- USACE Fremont (line B) 0.95 Confidence Limits (at Hwy 880)
- USACE Fremont (line B) 0.05 Confidence Limits (at Hwy 880)

Fremont Flood Control Channel, Alameda County, CA
Flow Frequency Curve Fremont Flood Control Channel at Highway 880
US Army Corps of Engineers San Francisco District

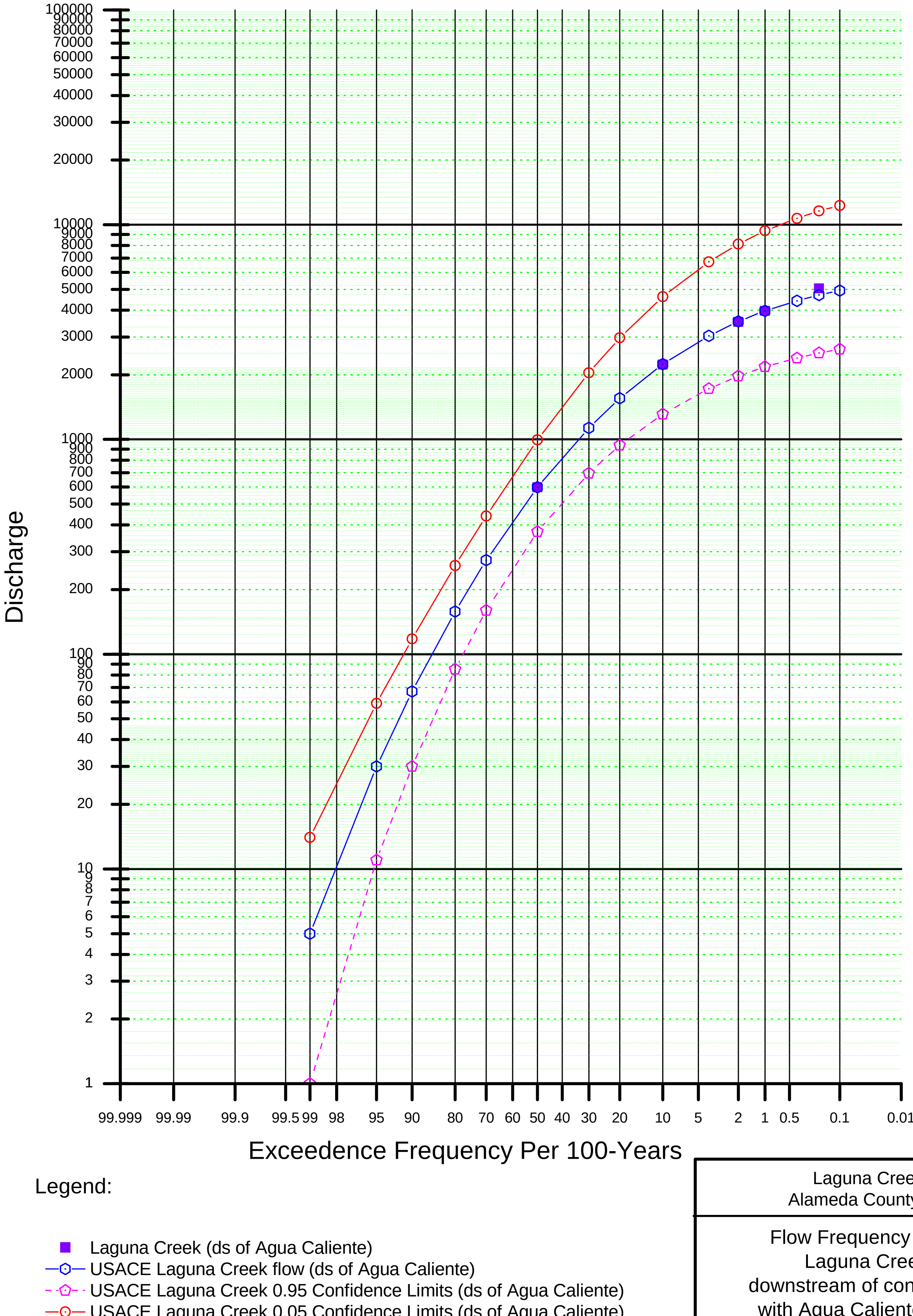


Plate 14

Laguna Creek,
Alameda County, CA

Flow Frequency Curve
Laguna Creek
downstream of confluence
with Agua Caliente Creek

US Army Corps of Engineers
San Francisco District

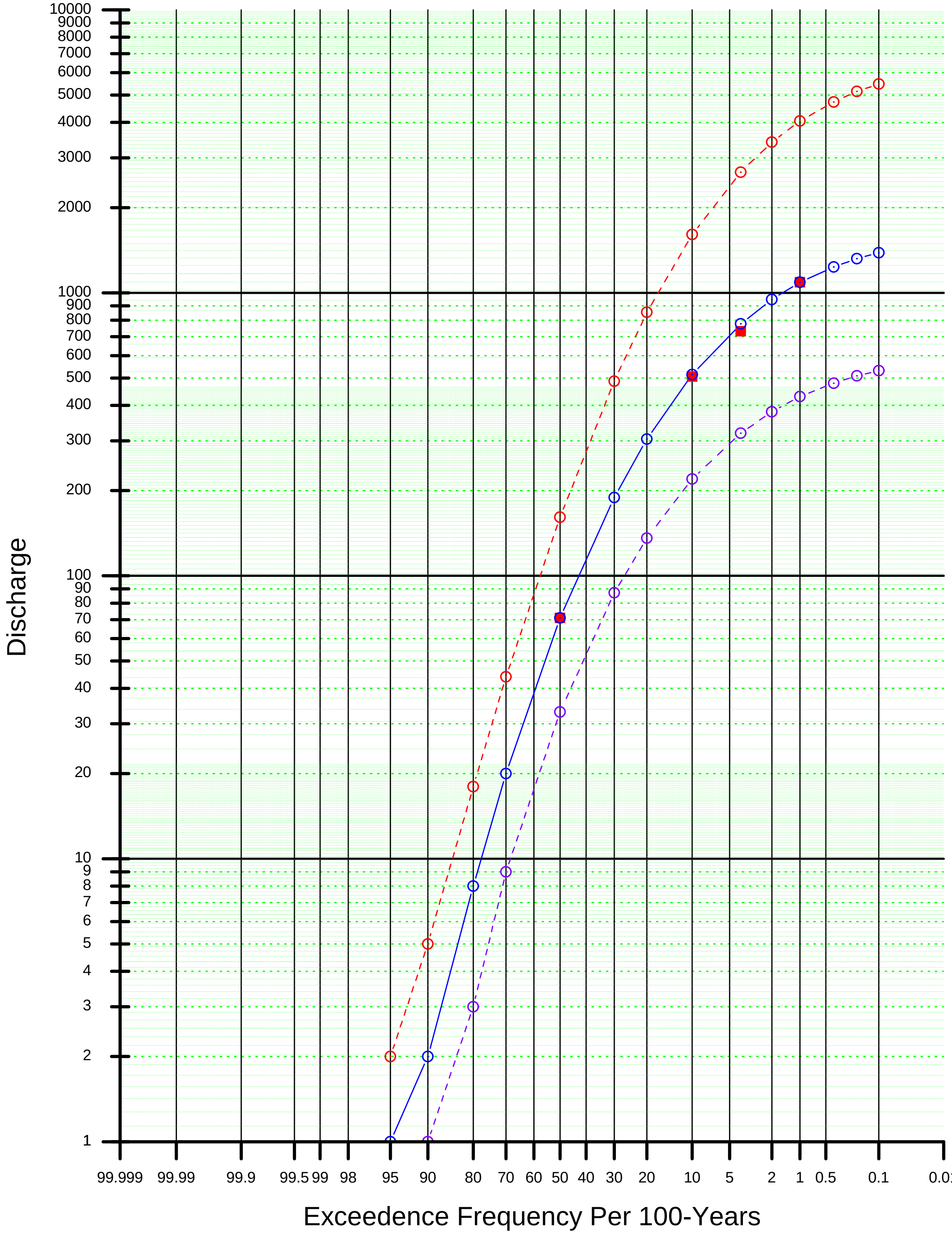


Plate 15

Legend:

- Agua Caliente (at Laguna)
- USACE Agua Caliente flow (at Laguna Creek)
- USACE Agua Caliente 0.95 Confidence Limits (at Laguna Creek)
- USACE Agua Caliente 0.05 Confidence Limits (at Laguna Creek)

Agua Caliente Creek,
Alameda County, CA

Flow Frequency Curve
Agua Caliente Creek
at Laguna Creek

US Army Corps of Engineers
San Francisco District

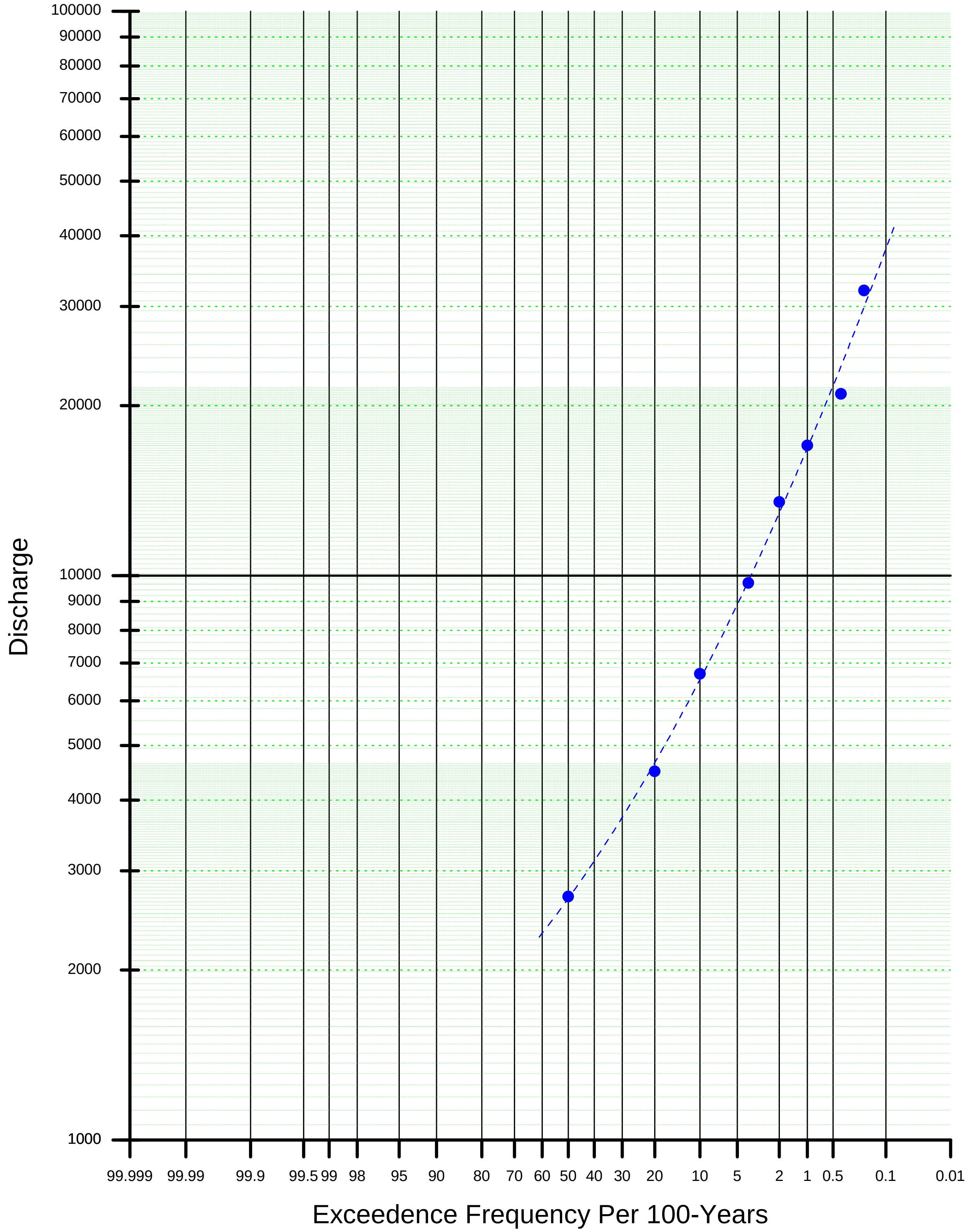


Plate 16

Legend:

USACE Guadalupe flow (at San Jose)

Guadalupe River, US Army Corps of Engineers
Flow Frequency Curve Guadalupe River at San Jose
US Army Corps of Engineers San Francisco District

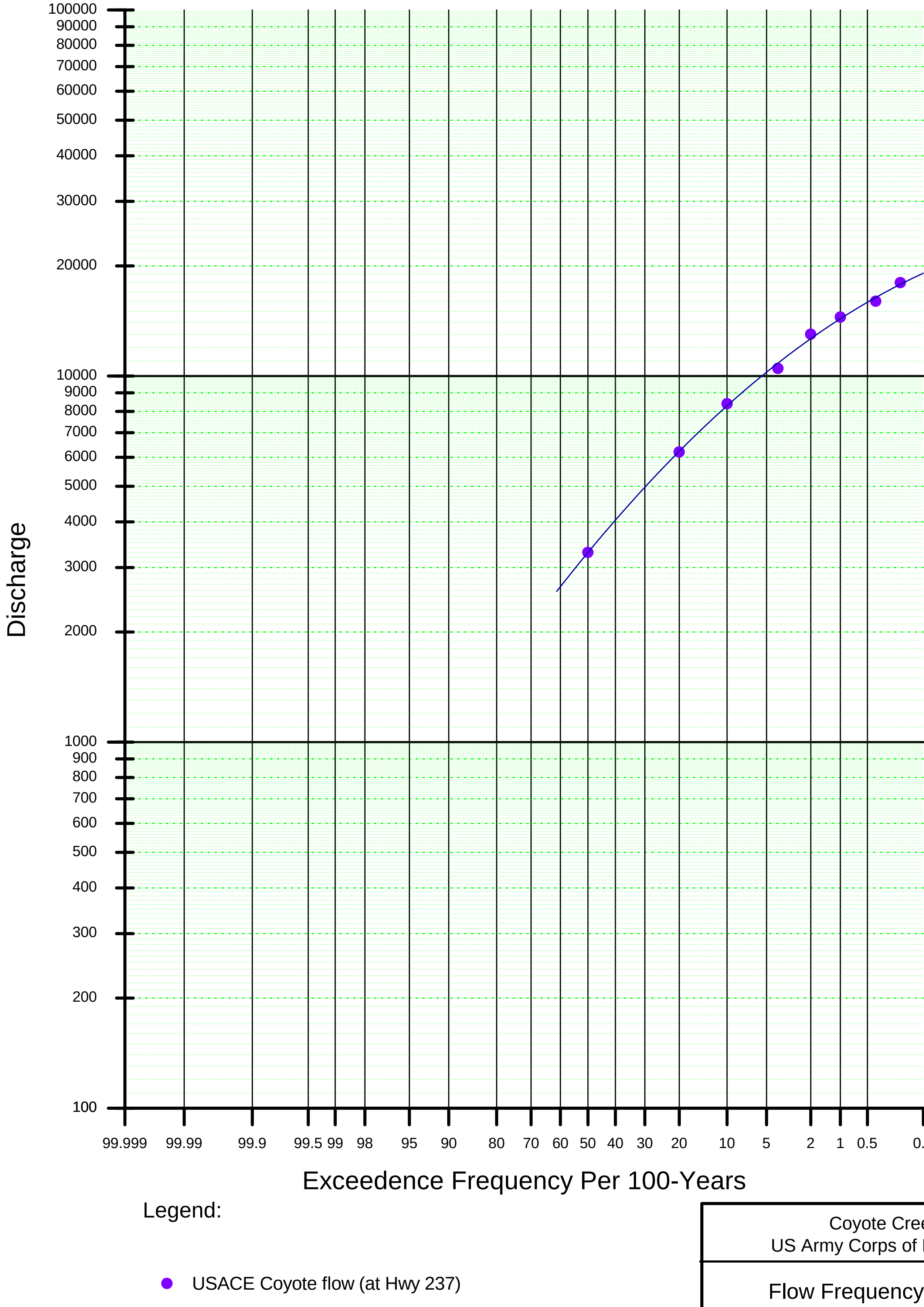


Plate 17

Coyote Creek,
US Army Corps of Engineers

Flow Frequency Curve
Coyote Creek
at Highway 237

US Army Corps of Engineers
San Francisco District

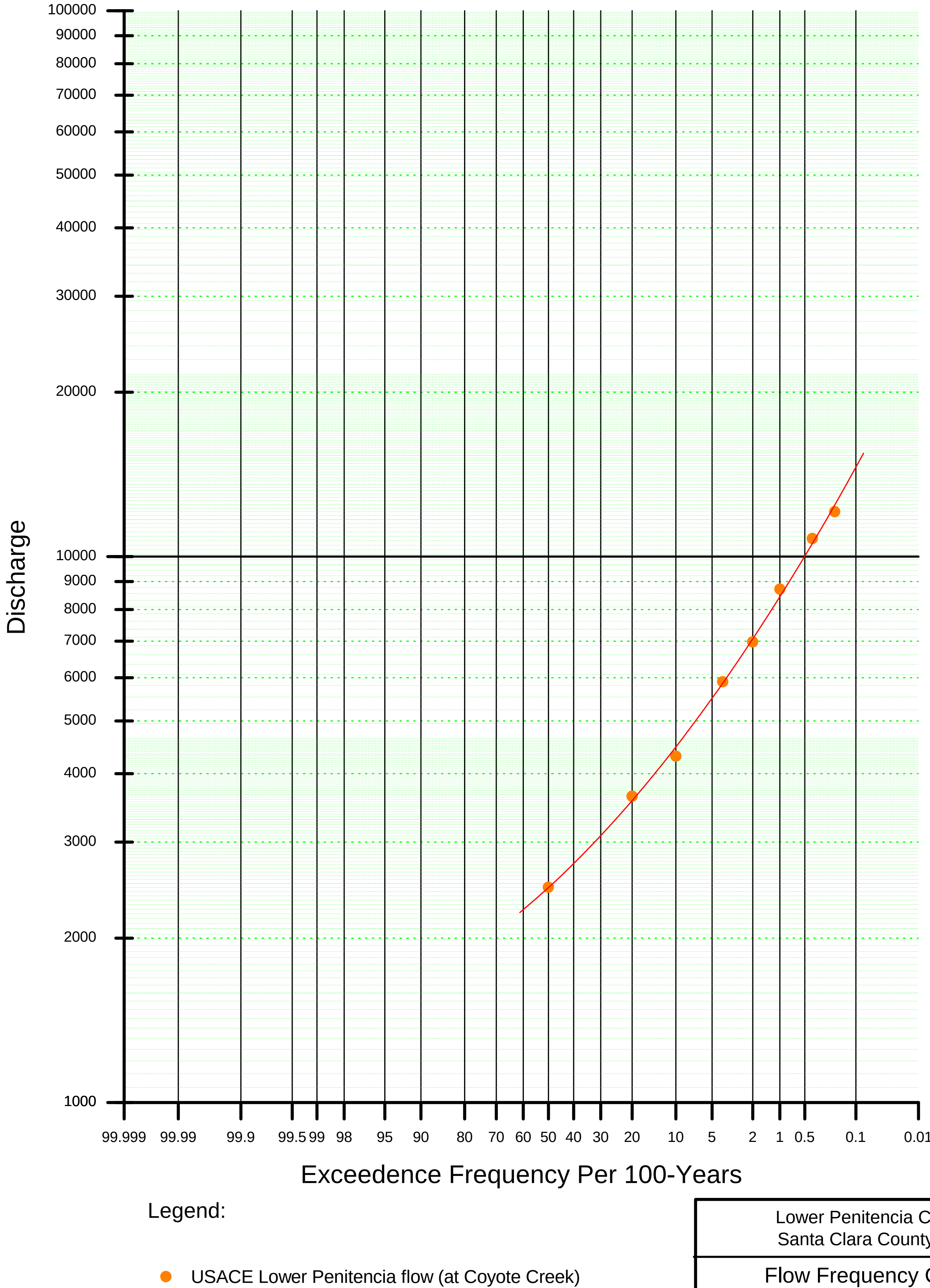
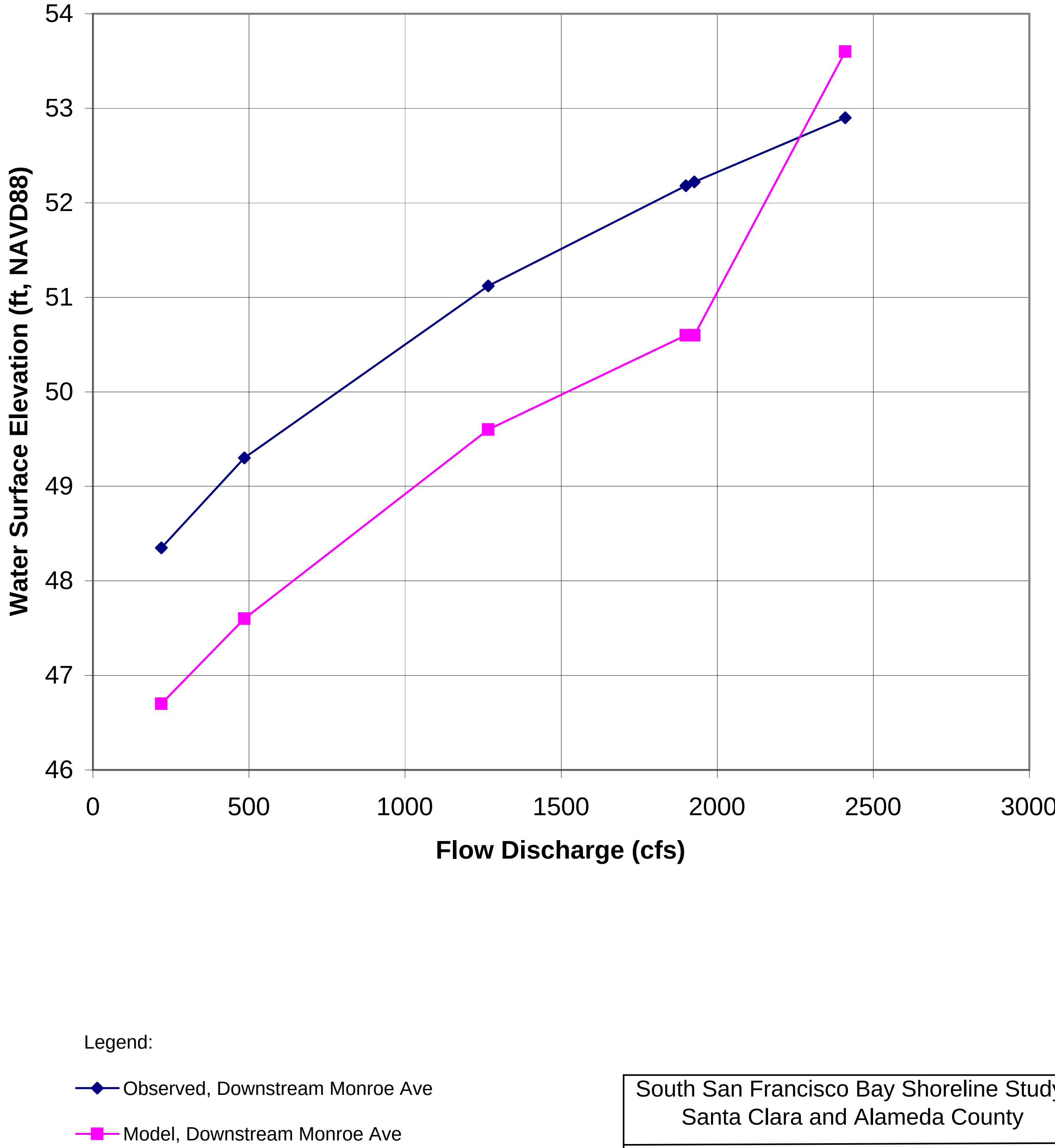


Plate 18

Lower Penitencia Creek,
Santa Clara County, CA

Flow Frequency Curve
Lower Penitencia Creek
at Coyote Creek

US Army Corps of Engineers
San Francisco District

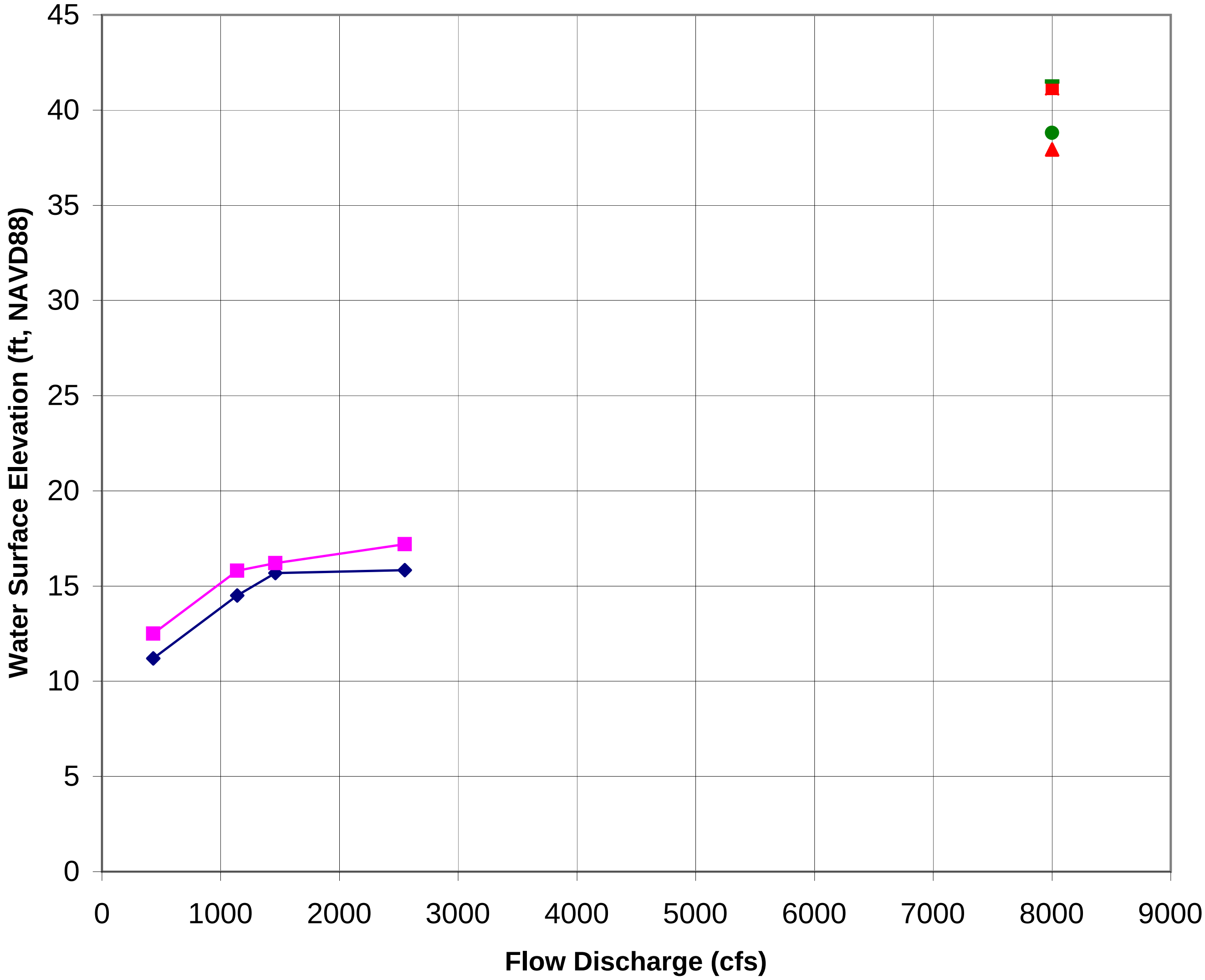


South San Francisco Bay Shoreline Study, Santa Clara and Alameda County

Calabazas Creek
Downstream Monroe Ave
Calibration Results

US Army Corps of Engineers
San Francisco District

PLATE 19



Legend:

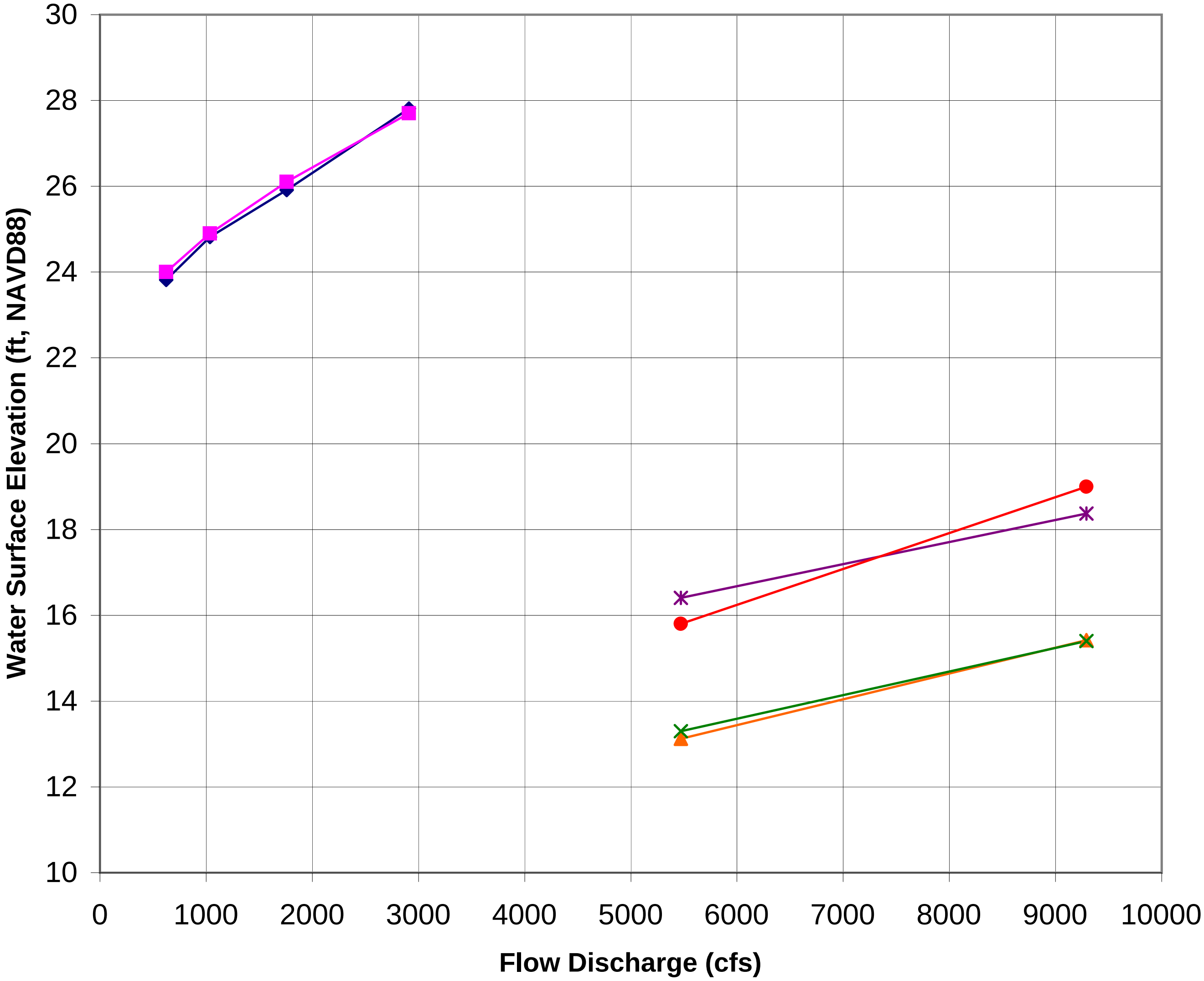
- Observed, Upstream Hwy 237
- Model, Upstream Hwy 237
- Observed, Upstream Charcot Ave
- Model, Upstream Charcot Ave
- Observed, Downstream Charcot Ave
- Model, Downstream Charcot Ave

South San Francisco Bay Shoreline Study, Santa Clara and Alameda County

Coyote Creek
Hwy 237 and Charcot Ave
Calibration Results

US Army Corps of Engineers
San Francisco District

PLATE 20



Legend:

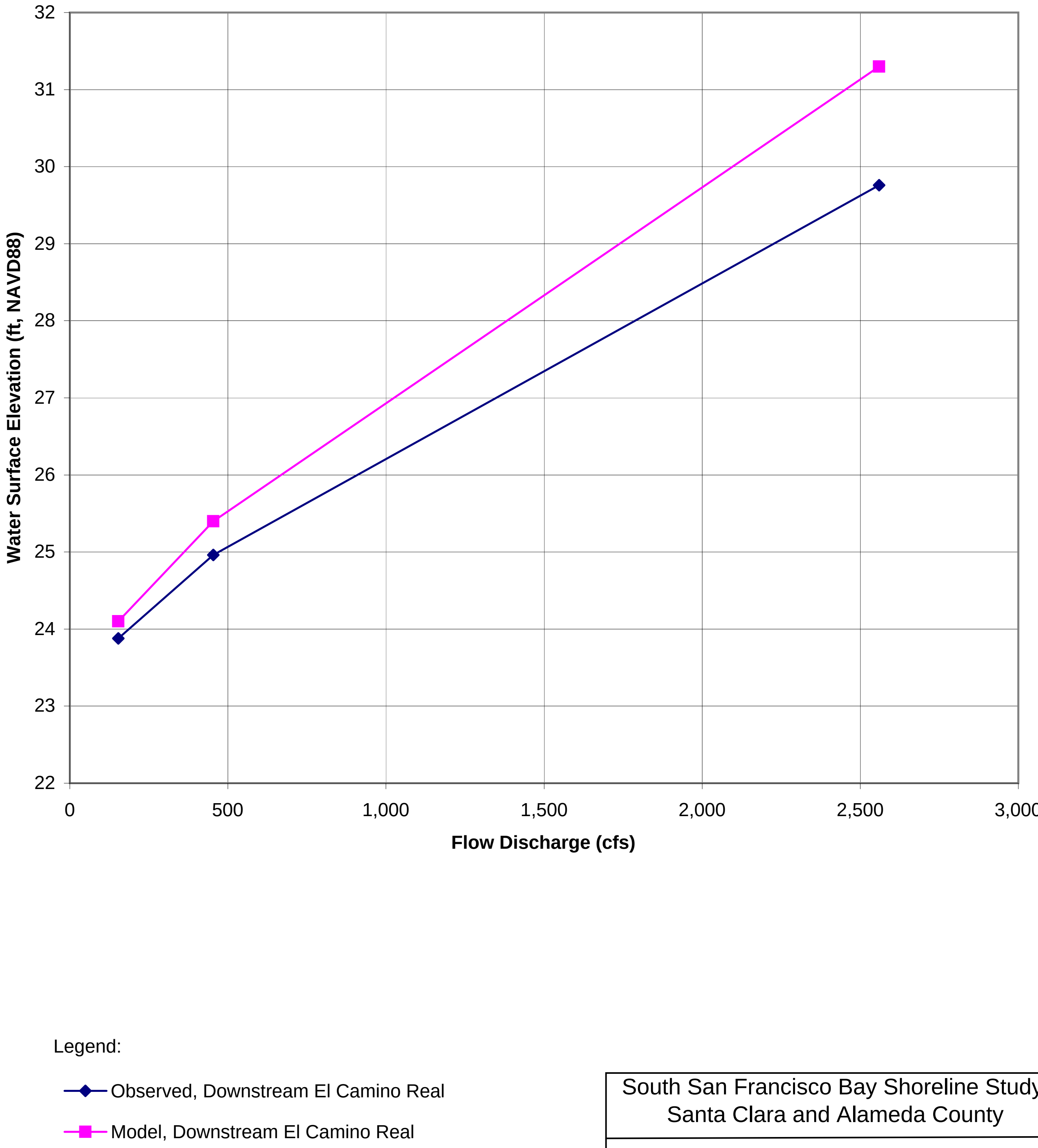
- Observed, Hwy 101
- Model, Hwy 101
- Observed, Upstream Hwy 237
- Model, Upstream Hwy 237
- Observed, Upstream Tasman Dr
- Model, Upstream Tasman Dr

South San Francisco Bay Shoreline Study,
Santa Clara and Alameda County

Guadalupe River
At Hwy101, Hwy 237, & Tasman Dr
Calibration Results

US Army Corps of Engineers
San Francisco District

PLATE 21

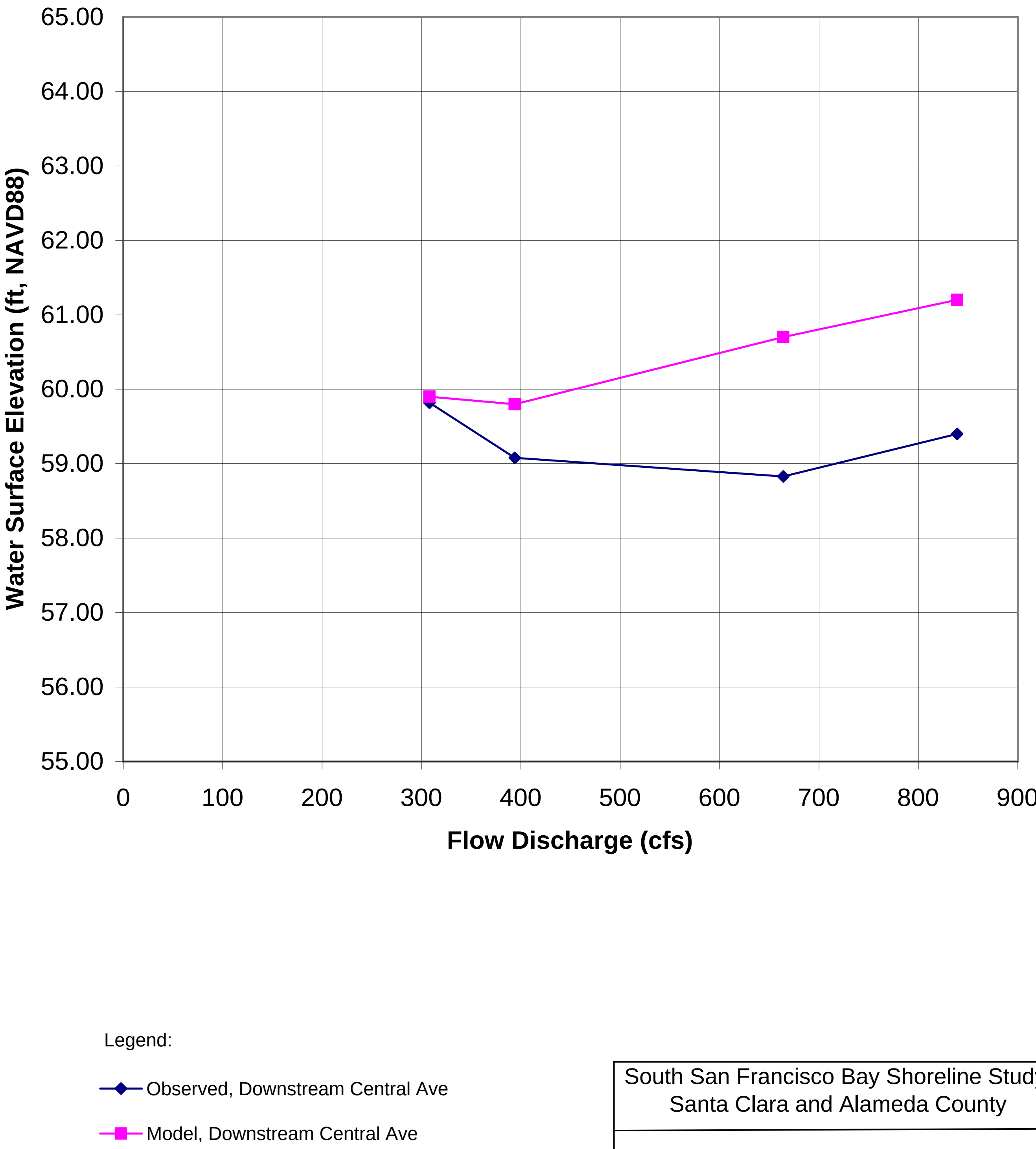


South San Francisco Bay Shoreline Study,
Santa Clara and Alameda County

Matadero Creek
At El Camino Real
Calibration Results

US Army Corps of Engineers
San Francisco District

PLATE 22

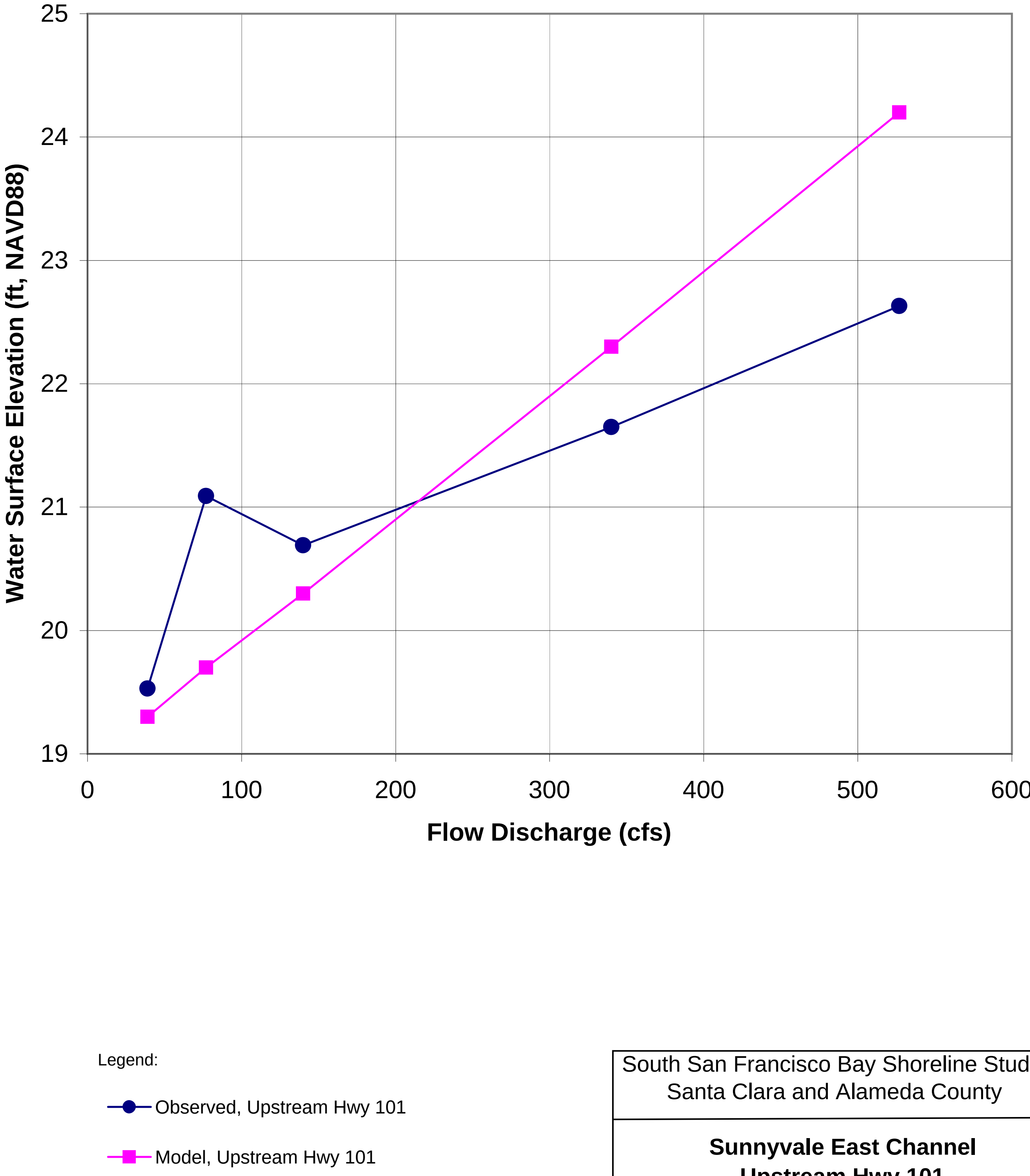


South San Francisco Bay Shoreline Study,
Santa Clara and Alameda County

Stevens Creek
Downstream Central Ave
Calibration Results

US Army Corps of Engineers
San Francisco District

PLATE 23

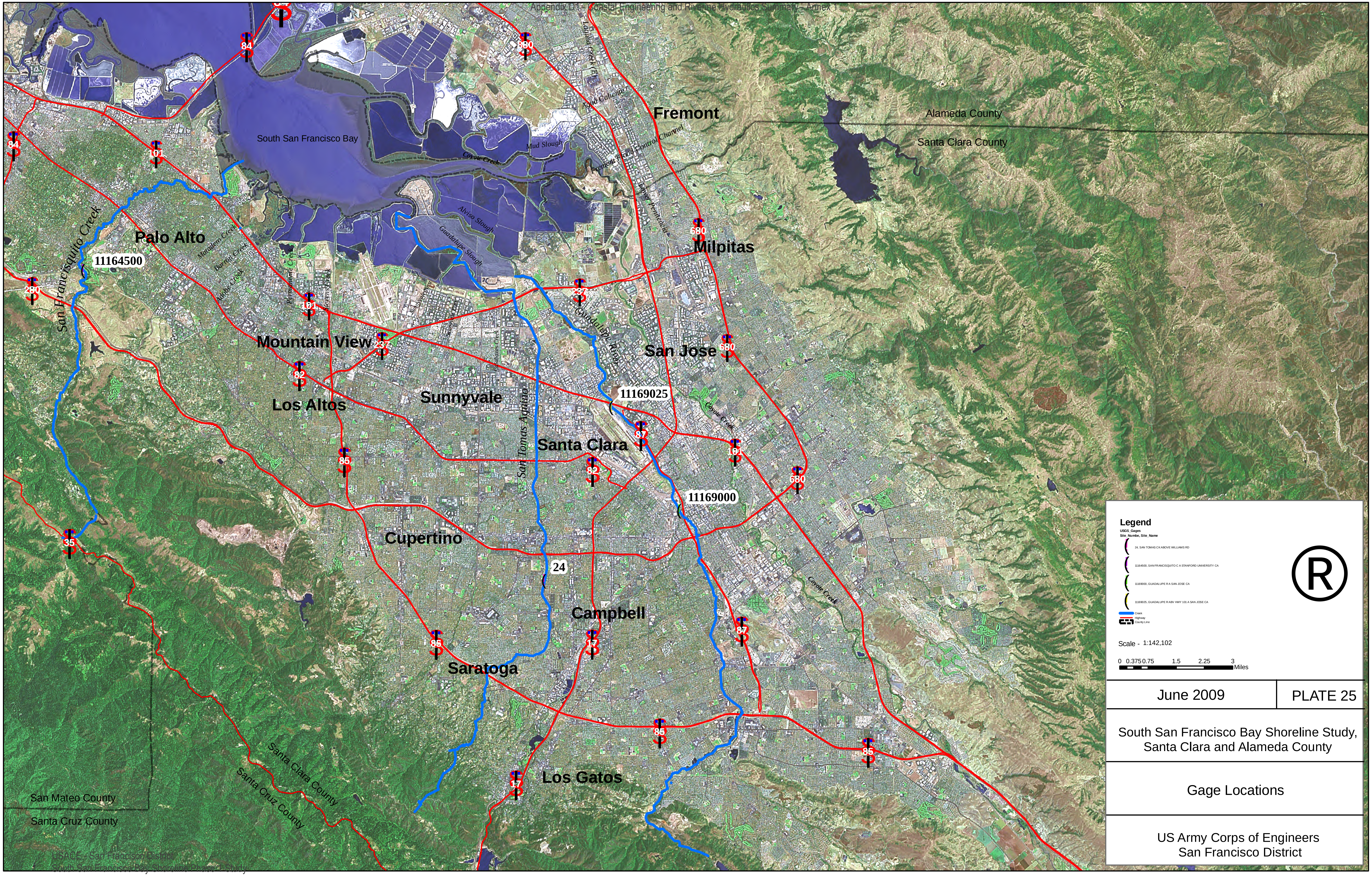


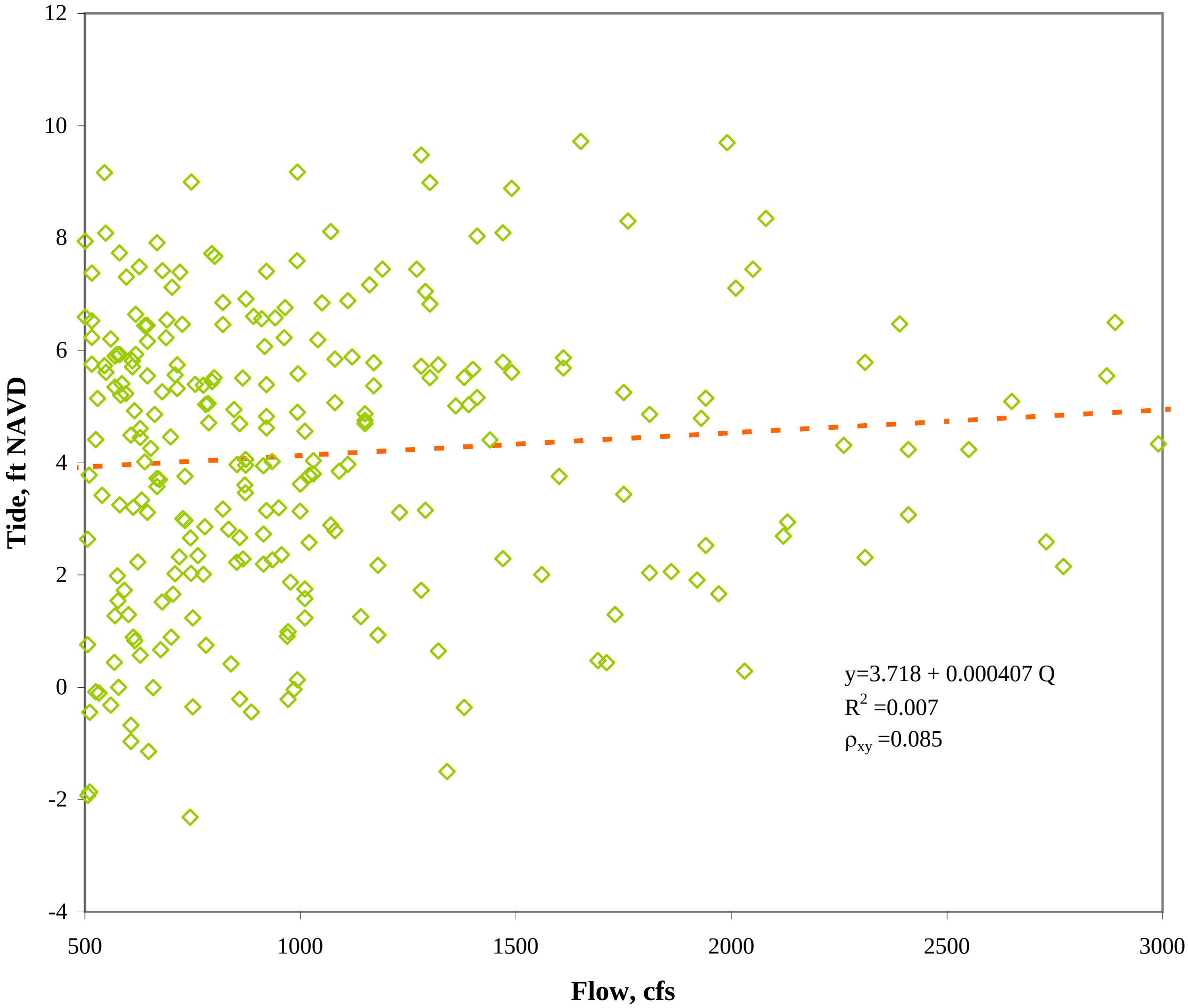
South San Francisco Bay Shoreline Study,
Santa Clara and Alameda County

Sunnyvale East Channel
Upstream Hwy 101
Calibration Results

US Army Corps of Engineers
San Francisco District

PLATE 24


South San Francisco Bay
Fremont
Alameda County
Santa Clara County
Milpitas
Palo Alto
11164500
San Francisquito Creek
Mountain View
San Jose
11169025
Sunnyvale
Los Altos
Santa Clara
11169000
San Tomas Aquino
Cupertino
24
Campbell
Saratoga
Los Gatos
Santa Clara County
Santa Cruz County
San Mateo County
Santa Cruz County
Alviso Slough
Guadalupe Slough
Guadalupe River
Coyote Creek
Mud Slough
Legend
Scale - 1:142,102
0 0.375 0.75 1.5 2.25 3
Miles
June 2009
PLATE 25
South San Francisco Bay Shoreline Study,
Santa Clara and Alameda County
Gage Locations
US Army Corps of Engineers
San Francisco District

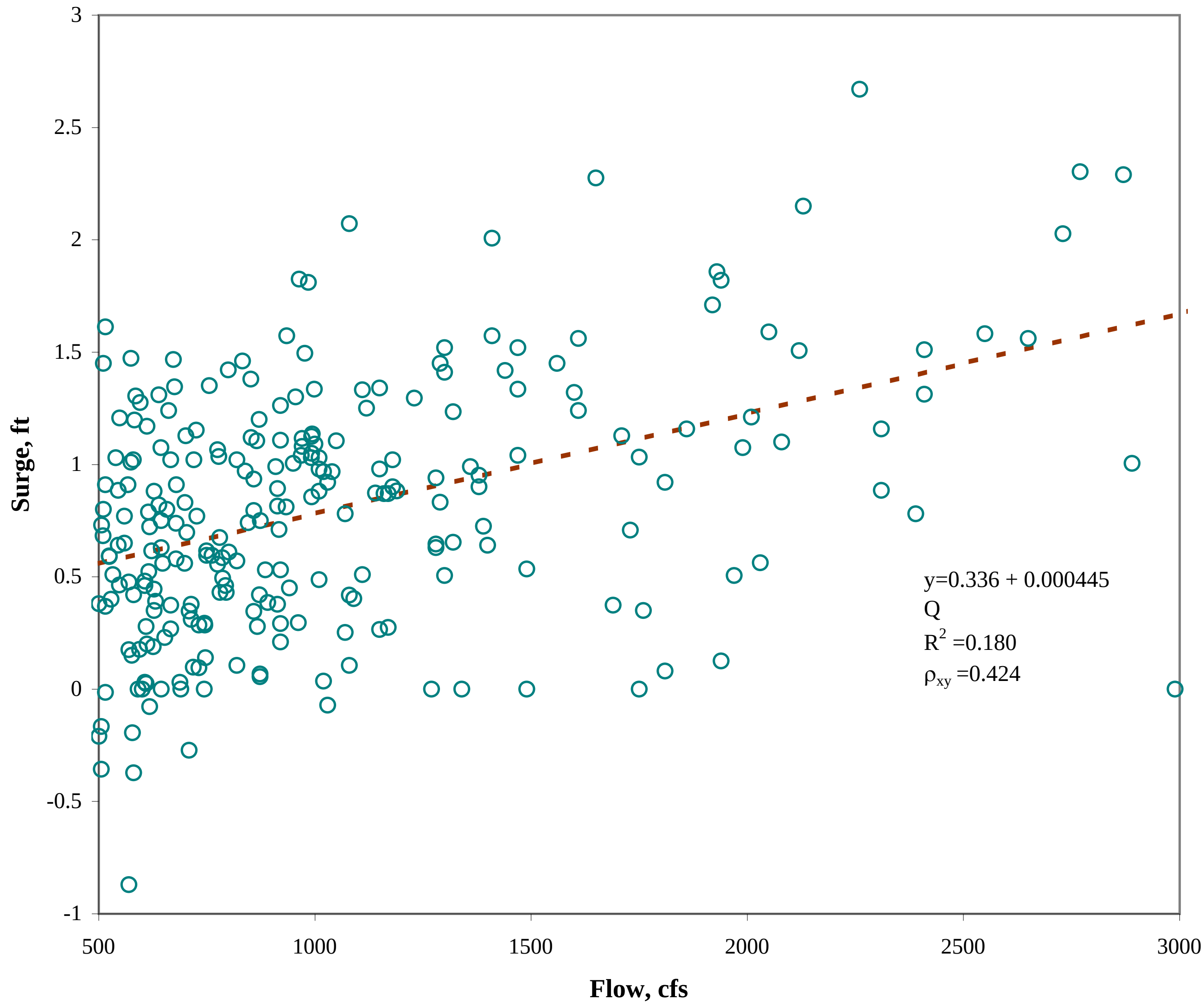


South San Francisco Bay Shoreline Study,
Santa Clara and Alameda County

San Thomas Aquino
Tide vs Peak Flow
Correlation Analysis

US Army Corps of Engineers
San Francisco District

Plate 26

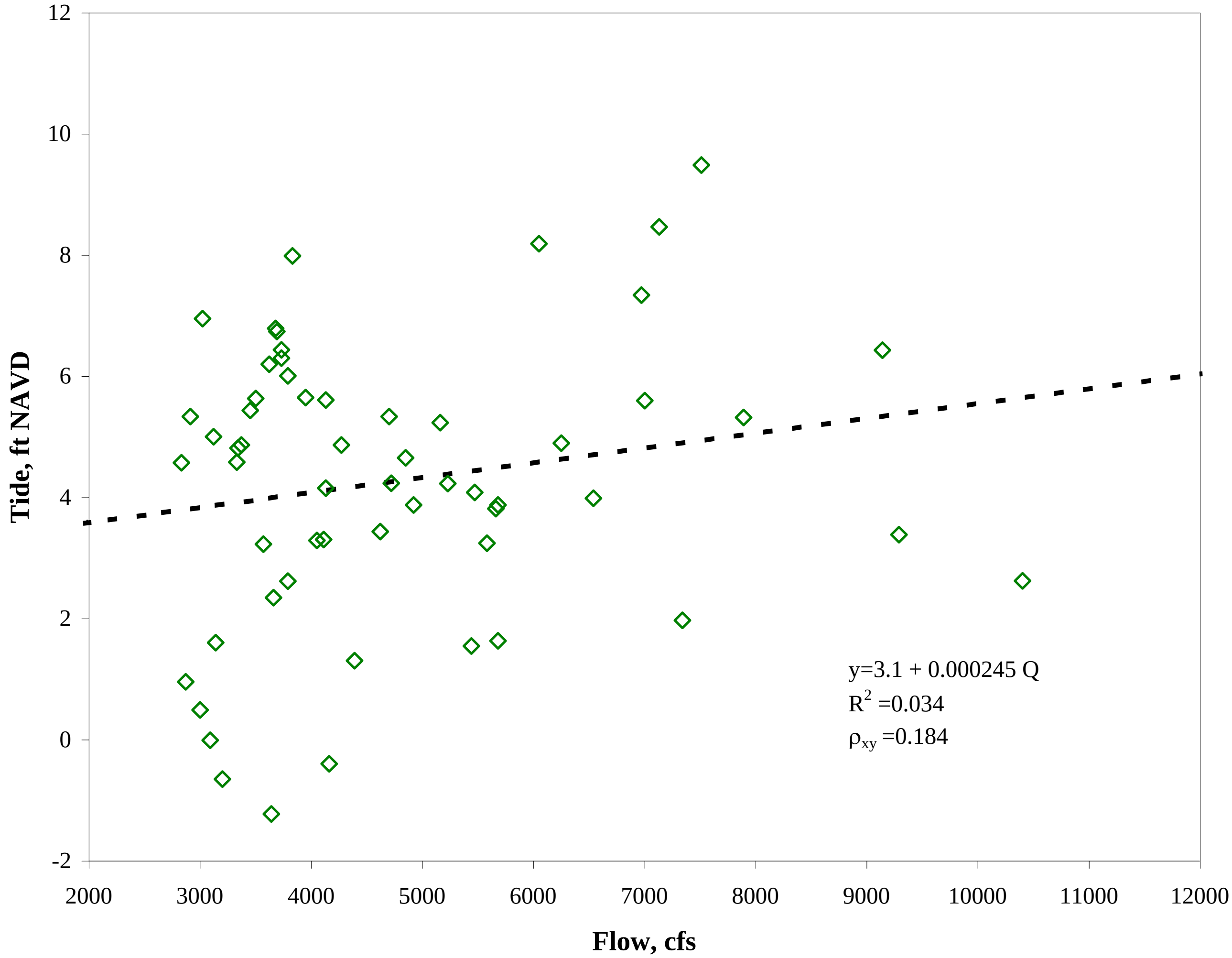


South San Francisco Bay Shoreline Study,
Santa Clara and Alameda County

San Thomas Aquino
Surge vs Peak Flow
Correlation Analysis

US Army Corps of Engineers
San Francisco District

Plate 27

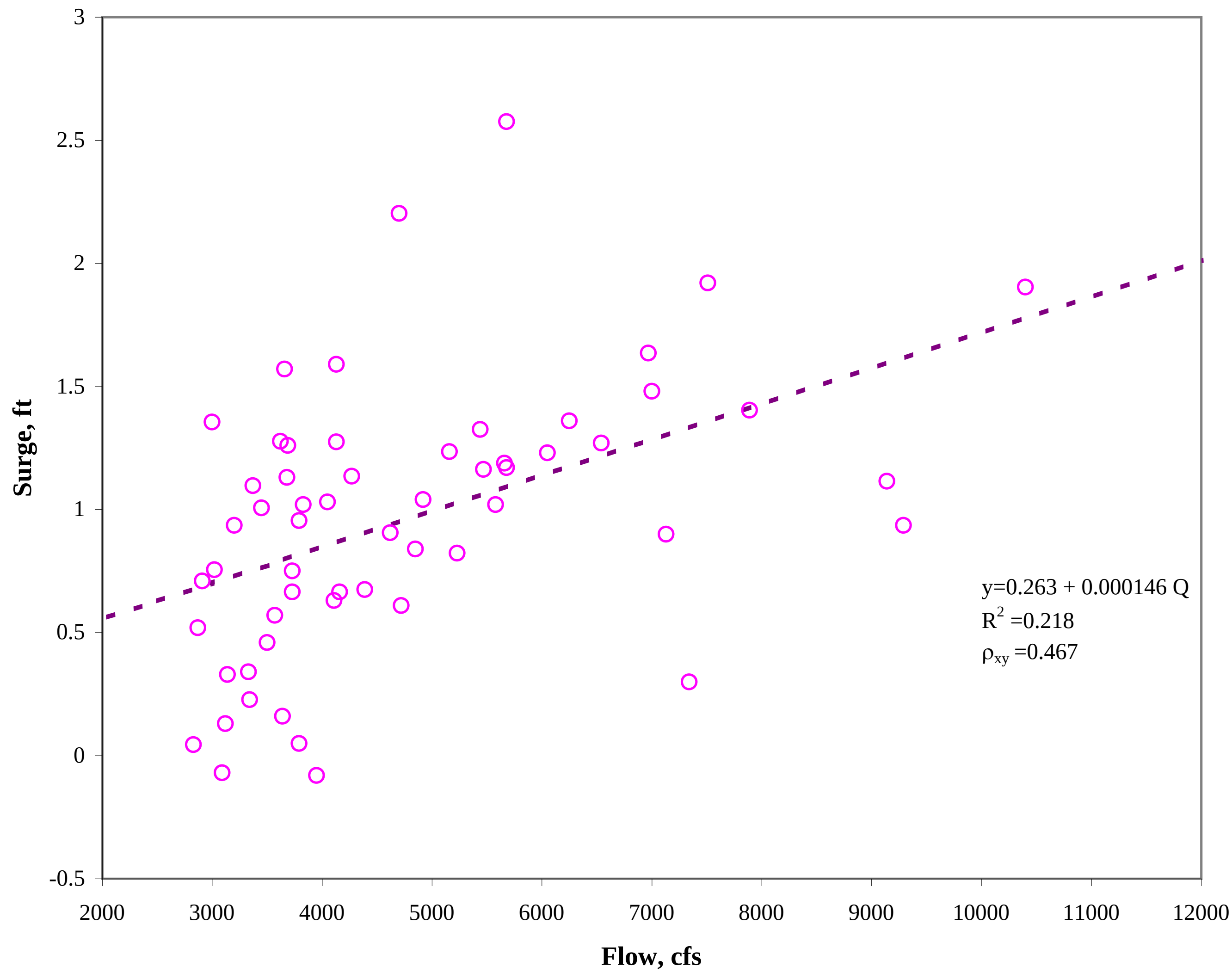


South San Francisco Bay Shoreline Study, Santa Clara and Alameda County

Guadalupe River Tide vs. Peak Flow Correlation Analysis

US Army Corps of Engineers
San Francisco District

PLATE 28

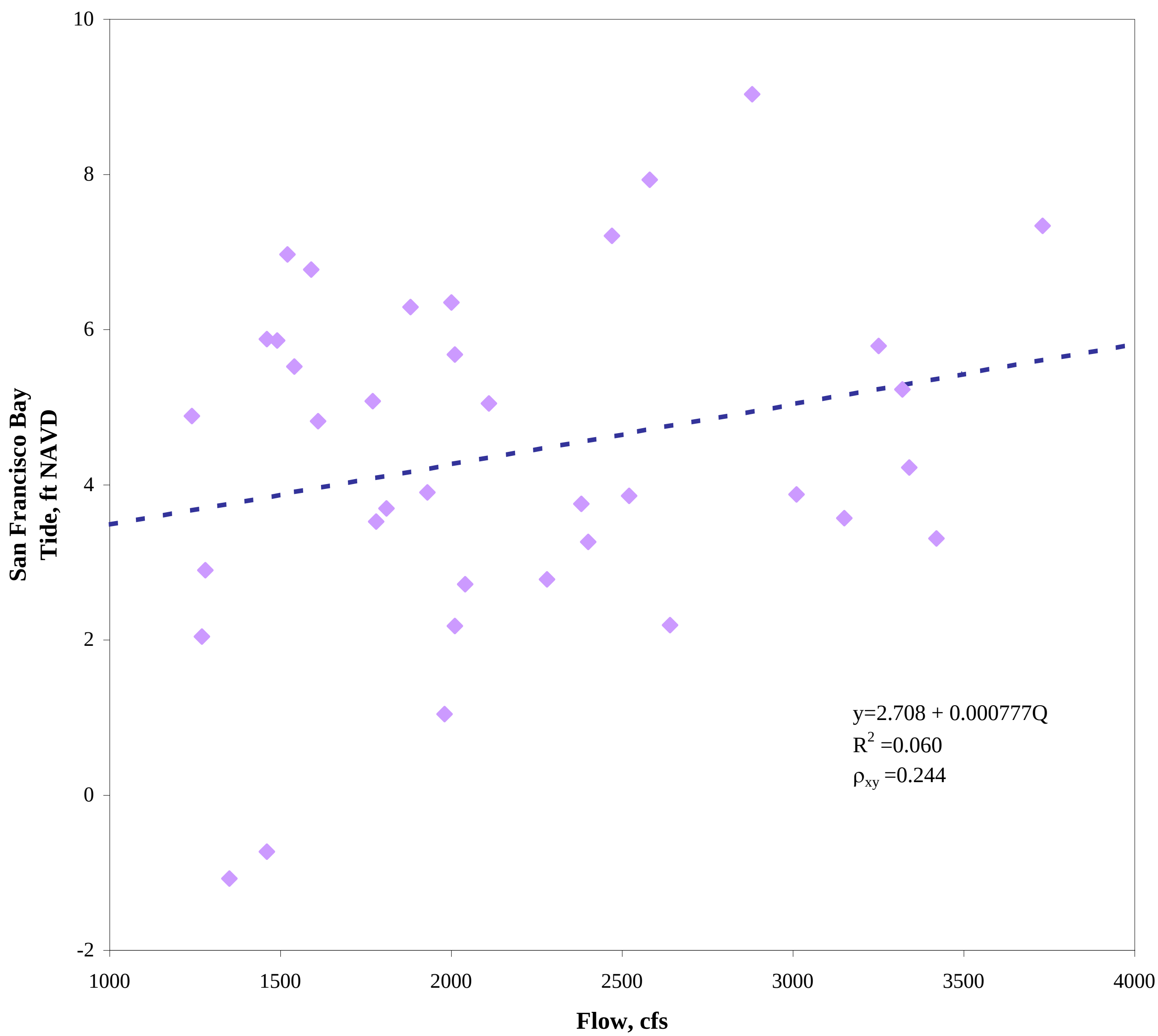


South San Francisco Bay Shoreline Study, Santa Clara and Alameda County

Guadalupe River
Surge vs. Peak Flow
Correlation Analysis

US Army Corps of Engineers
San Francisco District

PLATE 29

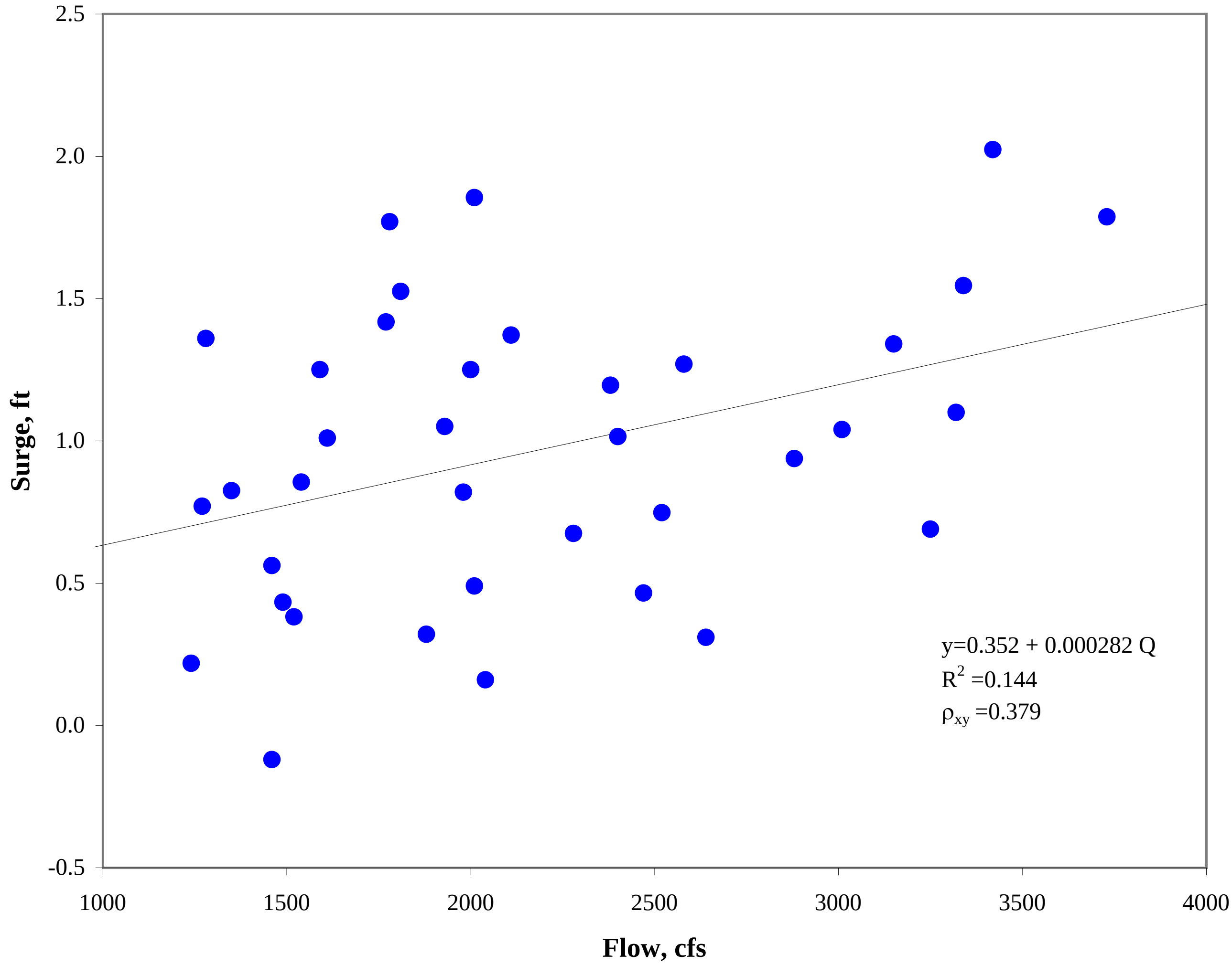


South San Francisco Bay Shoreline Study, Santa Clara and Alameda County

San Francisquito Creek
Tide vs. Peak Flow
Correlation Analysis

US Army Corps of Engineers
San Francisco District

PLATE 30

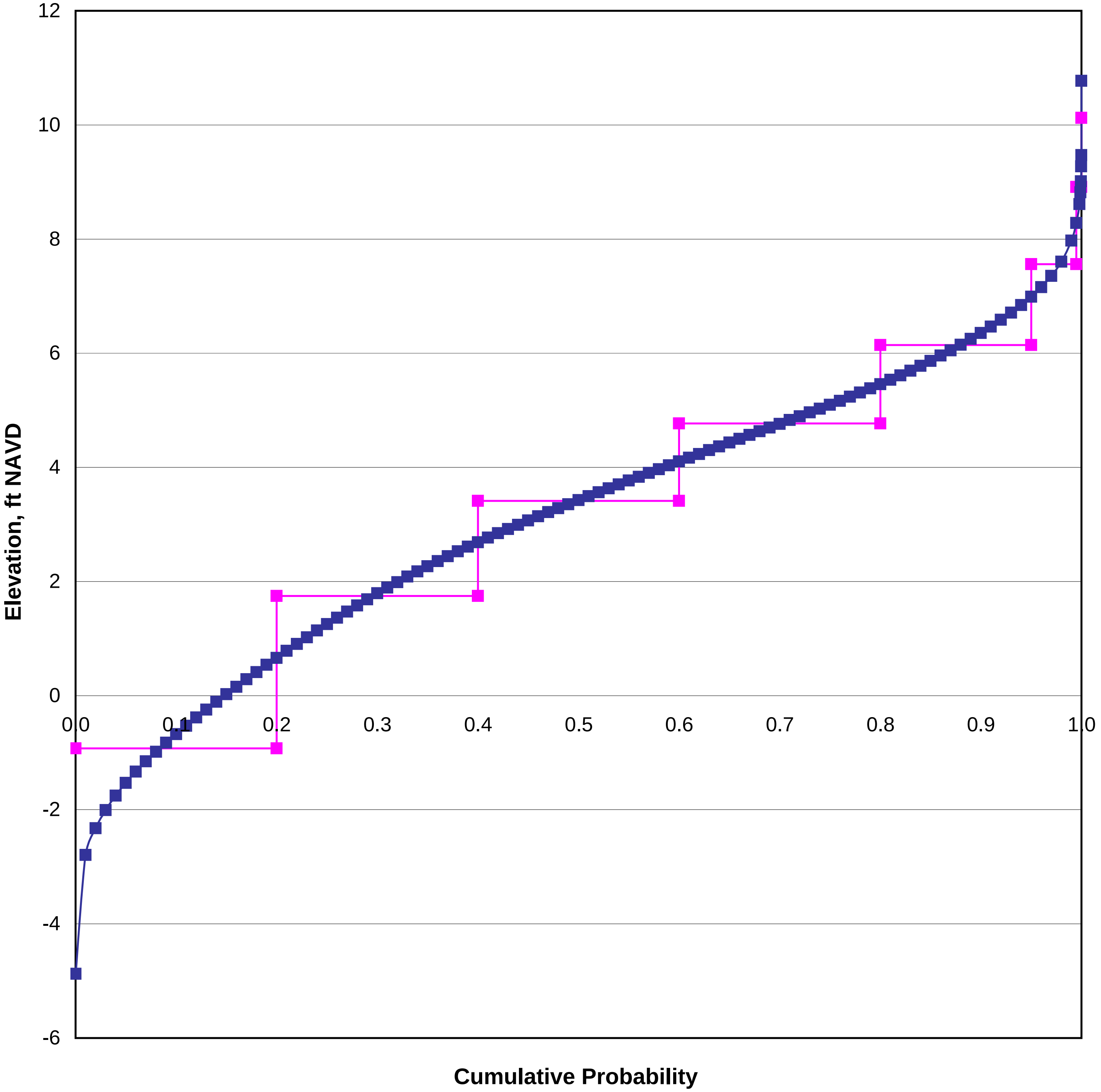


South San Francisco Bay Shoreline Study,
Santa Clara and Alameda County

San Francisquito Creek
Surge vs. Peak Flow
Correlation Analysis

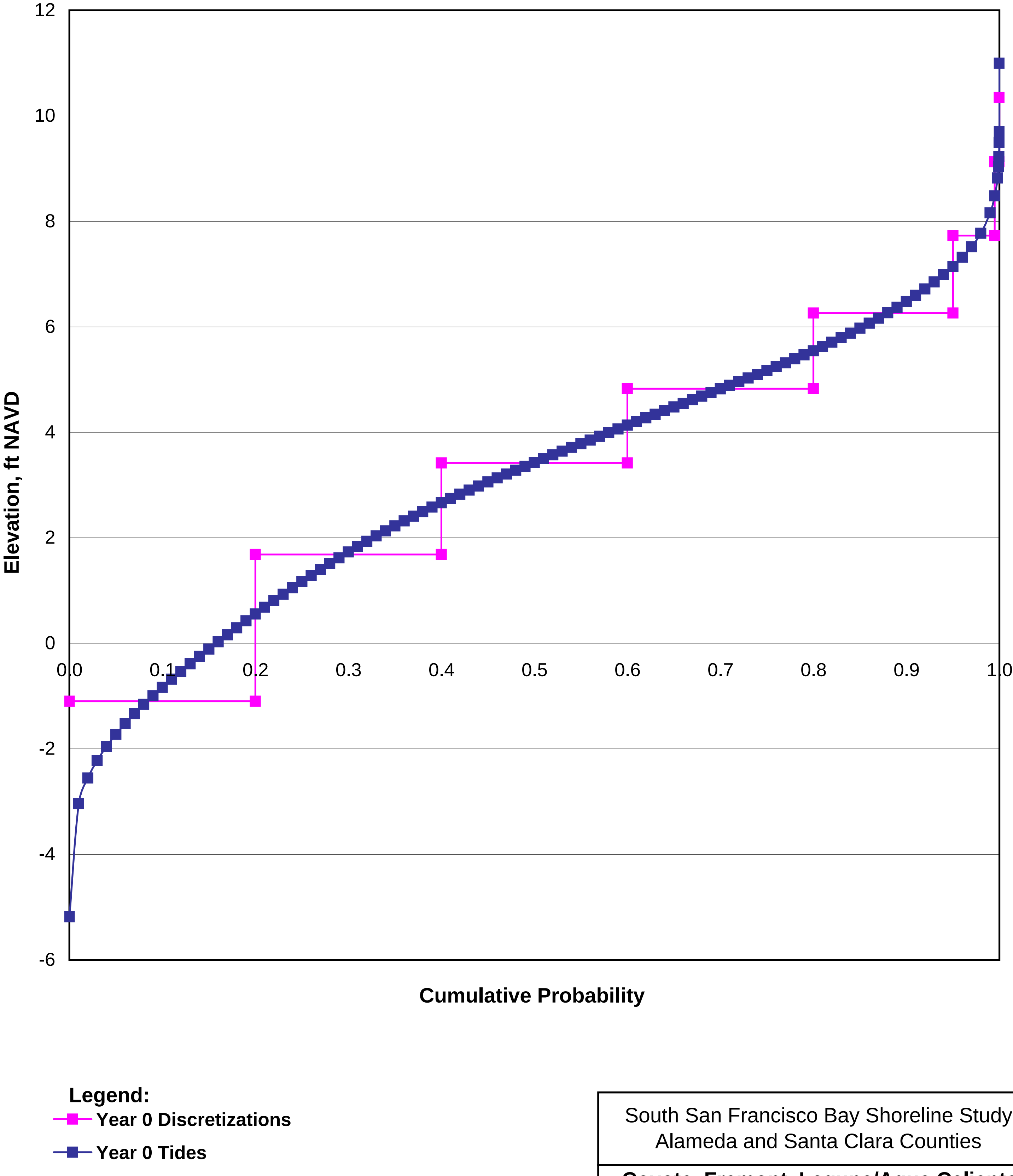
US Army Corps of Engineers
San Francisco District

PLATE 31



Legend:
- Year 0 Discretizations
- Year 0 Tides

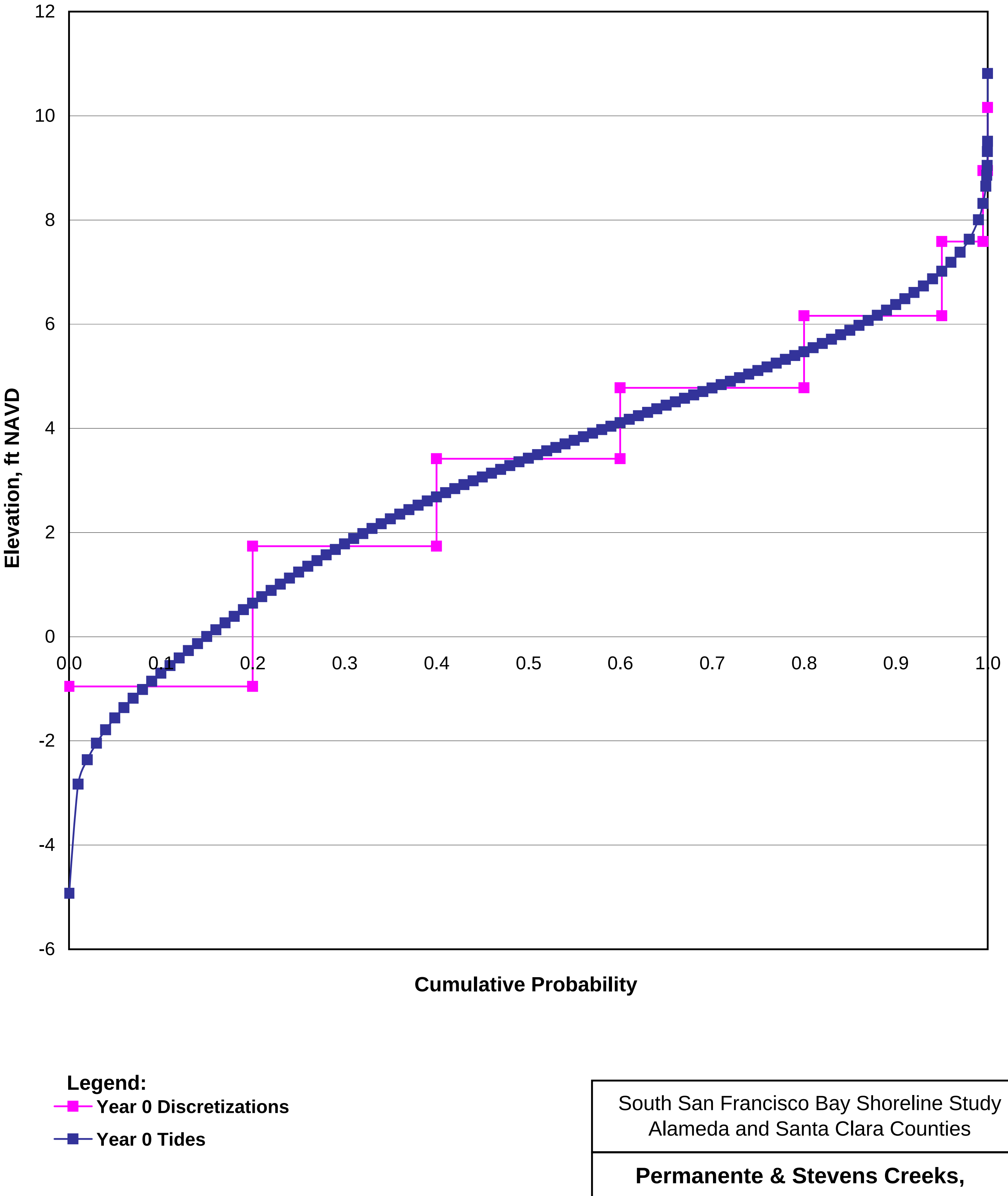
South San Francisco Bay Shoreline Study Alameda and Santa Clara Counties
Adobe, Barron & Matadero Creeks DS Boundary Conditions, Year 0 Tide Discretizations
US Army Corps of Engineers San Francisco District



South San Francisco Bay Shoreline Study
Alameda and Santa Clara Counties

Coyote, Fremont, Laguna/Aqua Caliente, Guadalupe & Lower Penitencia Creeks DS Boundary Conditions, Year 0 Tide Discretizations

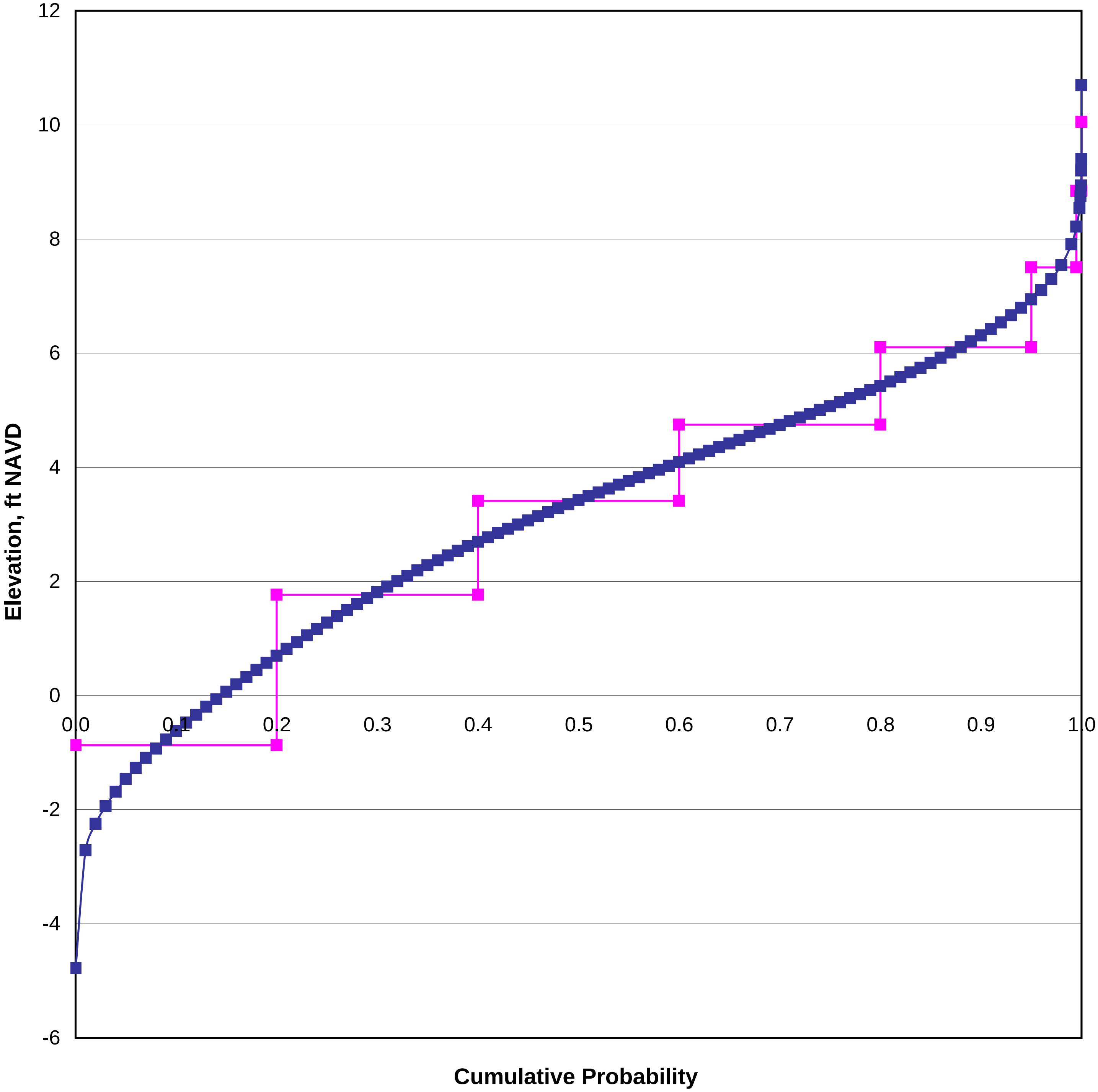
US Army Corps of Engineers
San Francisco District



South San Francisco Bay Shoreline Study
Alameda and Santa Clara Counties

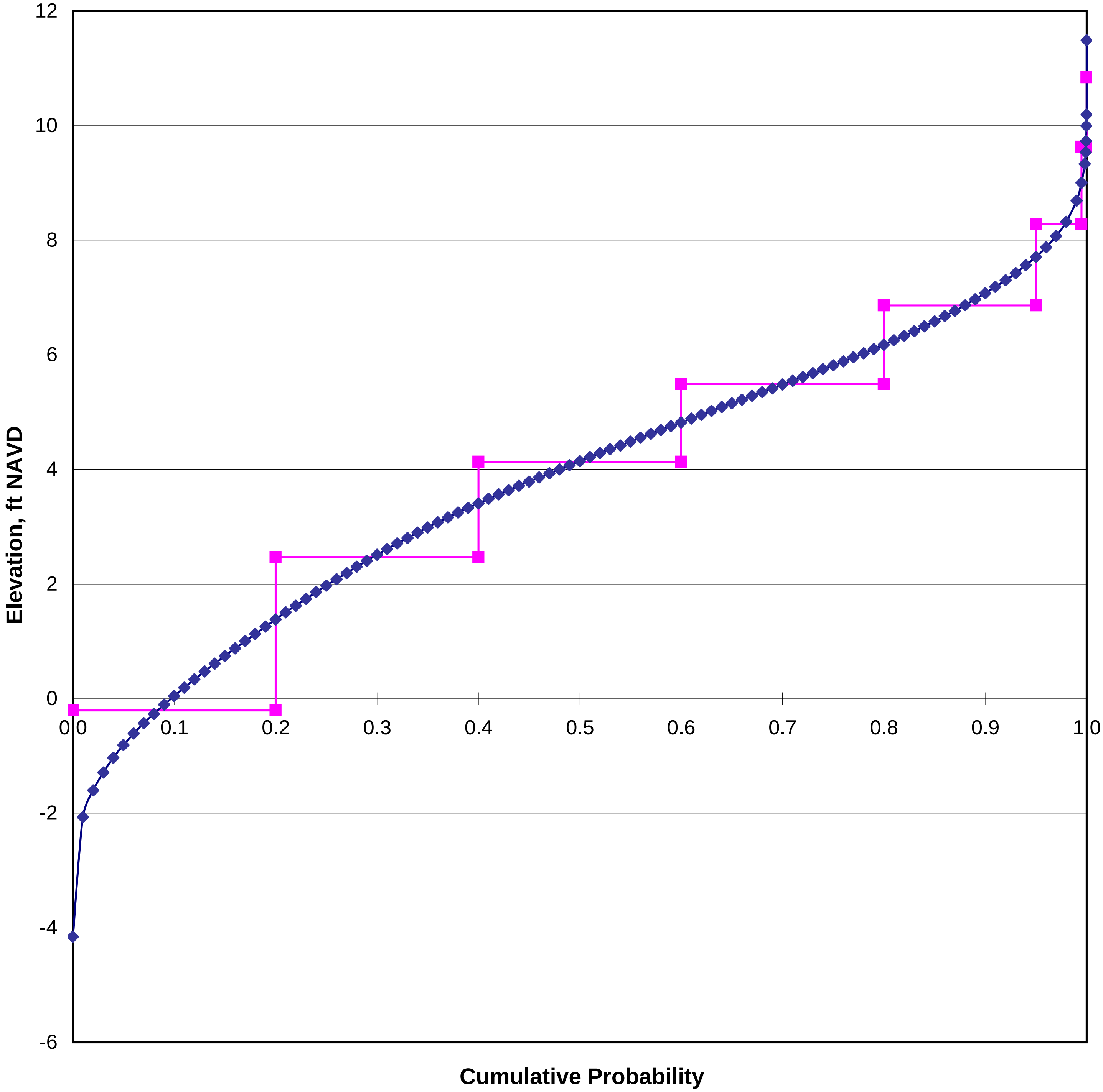
Permanente & Stevens Creeks, DS Boundary Conditions, Year 0 Tide Discretizations

US Army Corps of Engineers
San Francisco District



Legend:
- Year 0 Discretizations
- Year 0 Tides

South San Francisco Bay Shoreline Study Alameda and Santa Clara Counties
Guadalupe Slough & Sunnyvale West Channel, DS Boundary Conditions, Year 0 Tide Discretizations
US Army Corps of Engineers San Francisco District



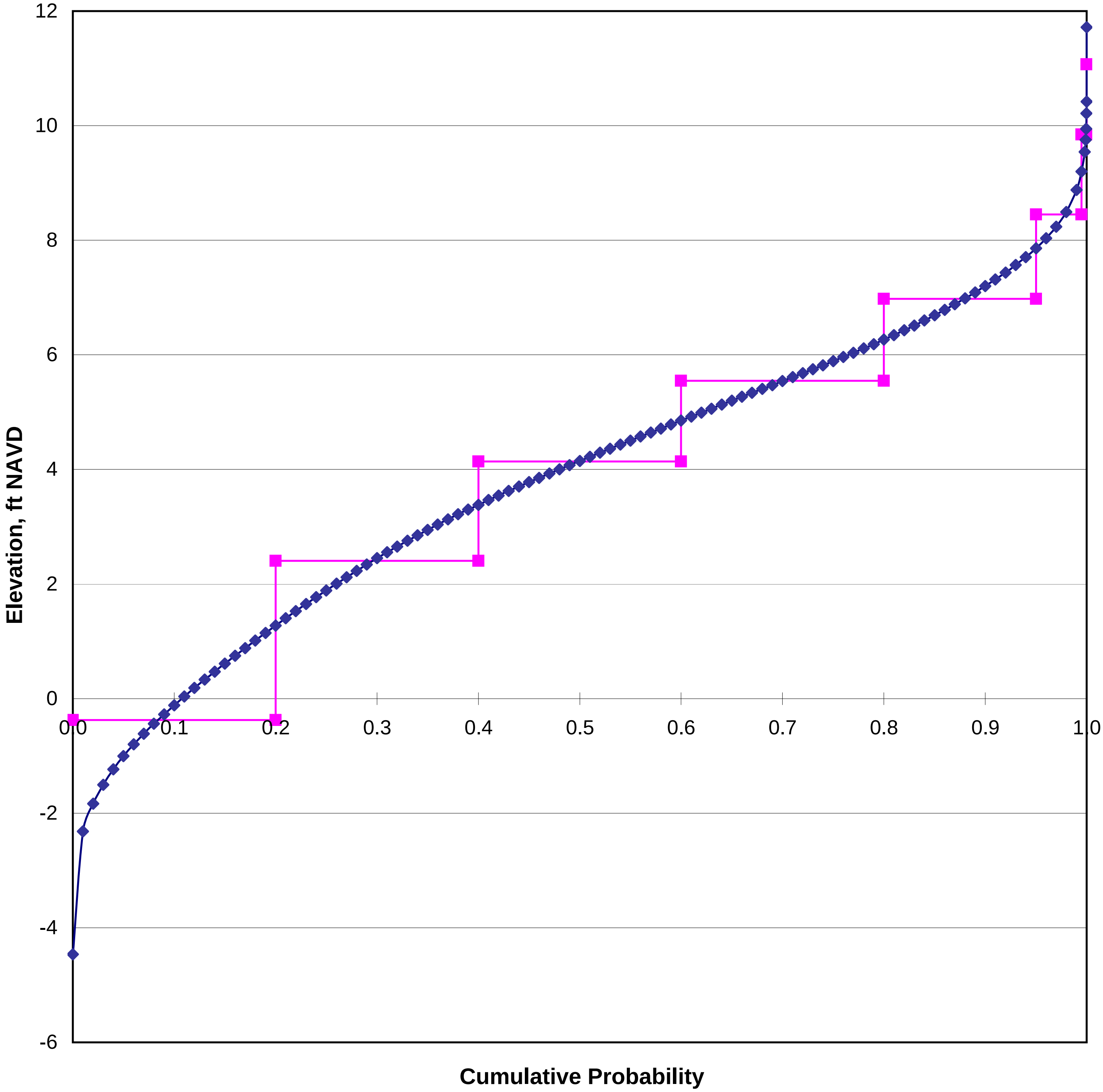
Legend:

- Year 50 Discretizations
- Year 50 Tides with 0.72 ft of Sea Level Rise

South San Francisco Bay Shoreline Study
Alameda and Santa Clara Counties

Adobe, Barron & Matadero Creeks DS Boundary Conditions, Year 50 Tide Discretizations

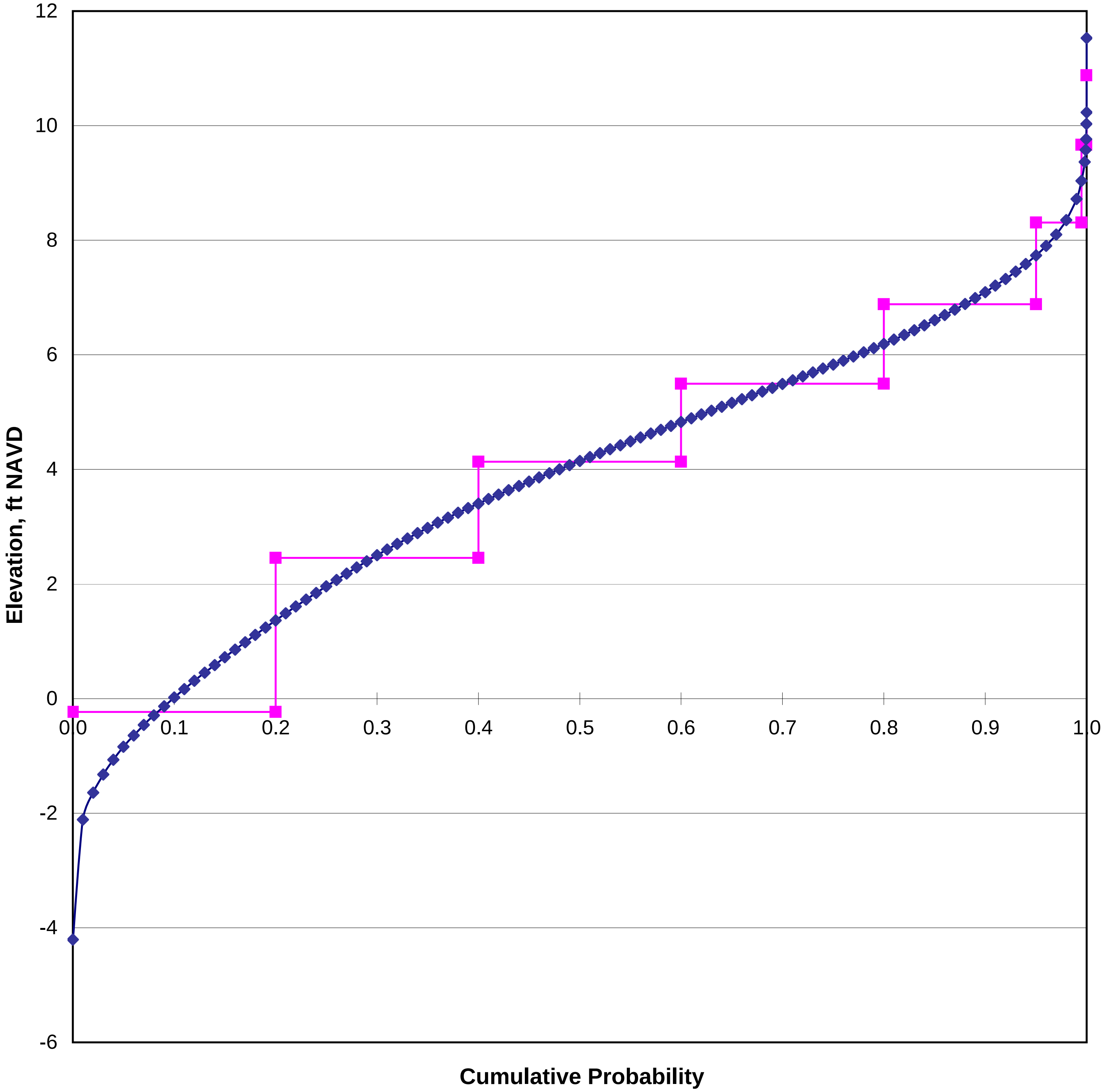
US Army Corps of Engineers
San Francisco District



Legend:

- Year 50 Discretizations
- Year 50 Tides with 0.72 ft of Sea Level Rise

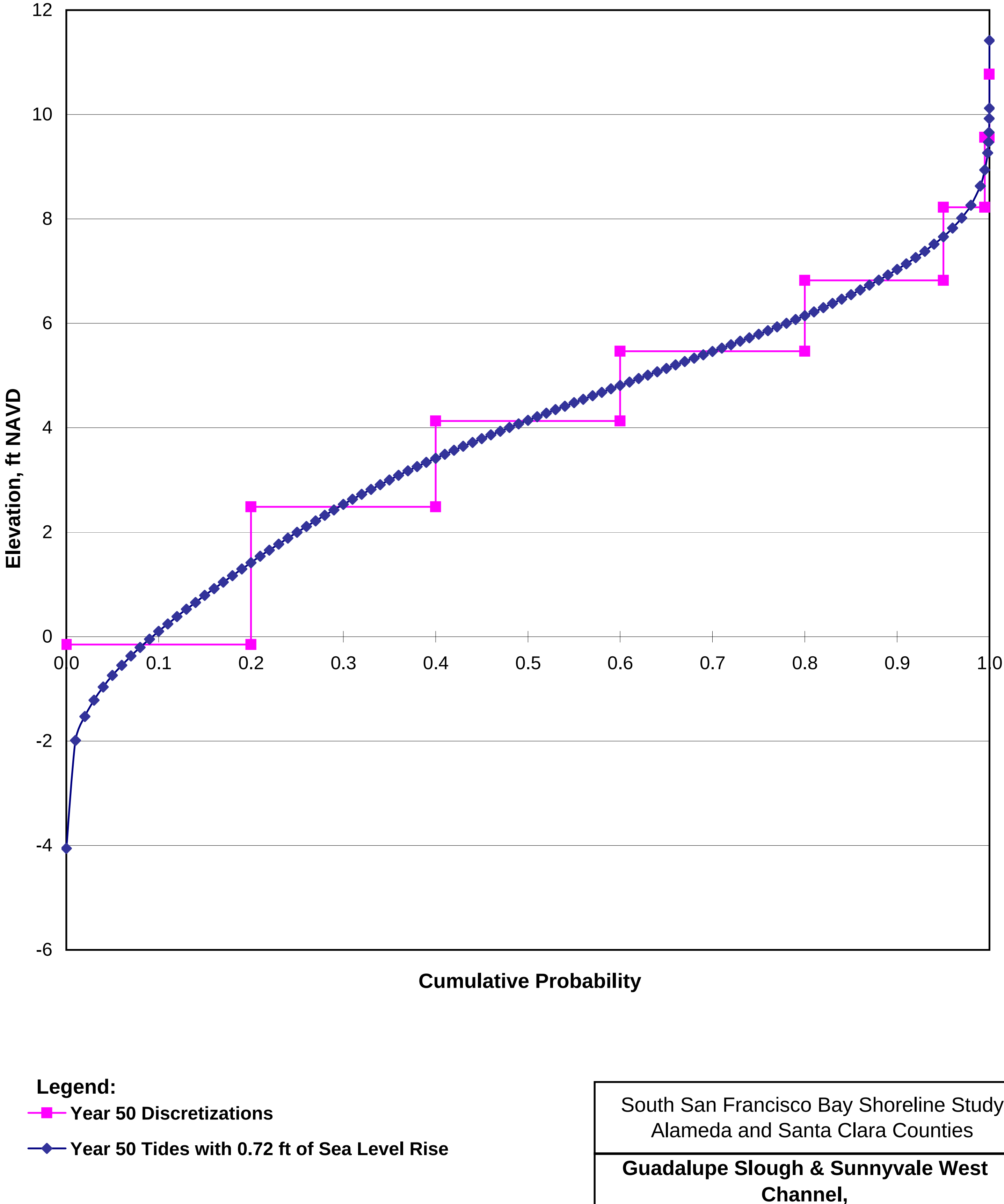
South San Francisco Bay Shoreline Study Alameda and Santa Clara Counties
Coyote, Fremont, Laguna/Aqua Caliente, Guadalupe & Lower Penitencia Creeks DS Boundary Conditions, Year 50 Tide Discretizations
US Army Corps of Engineers San Francisco District



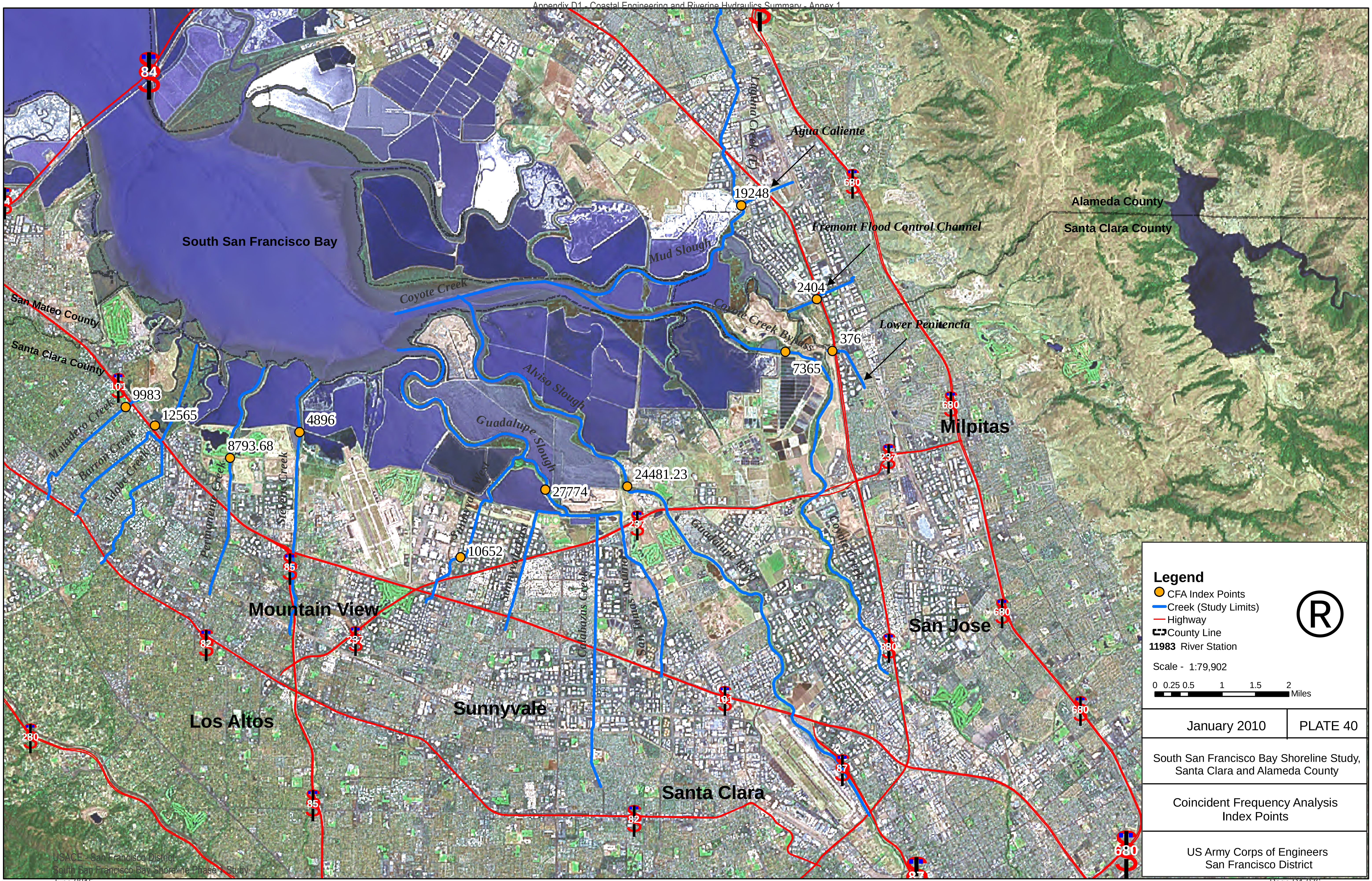
Legend:

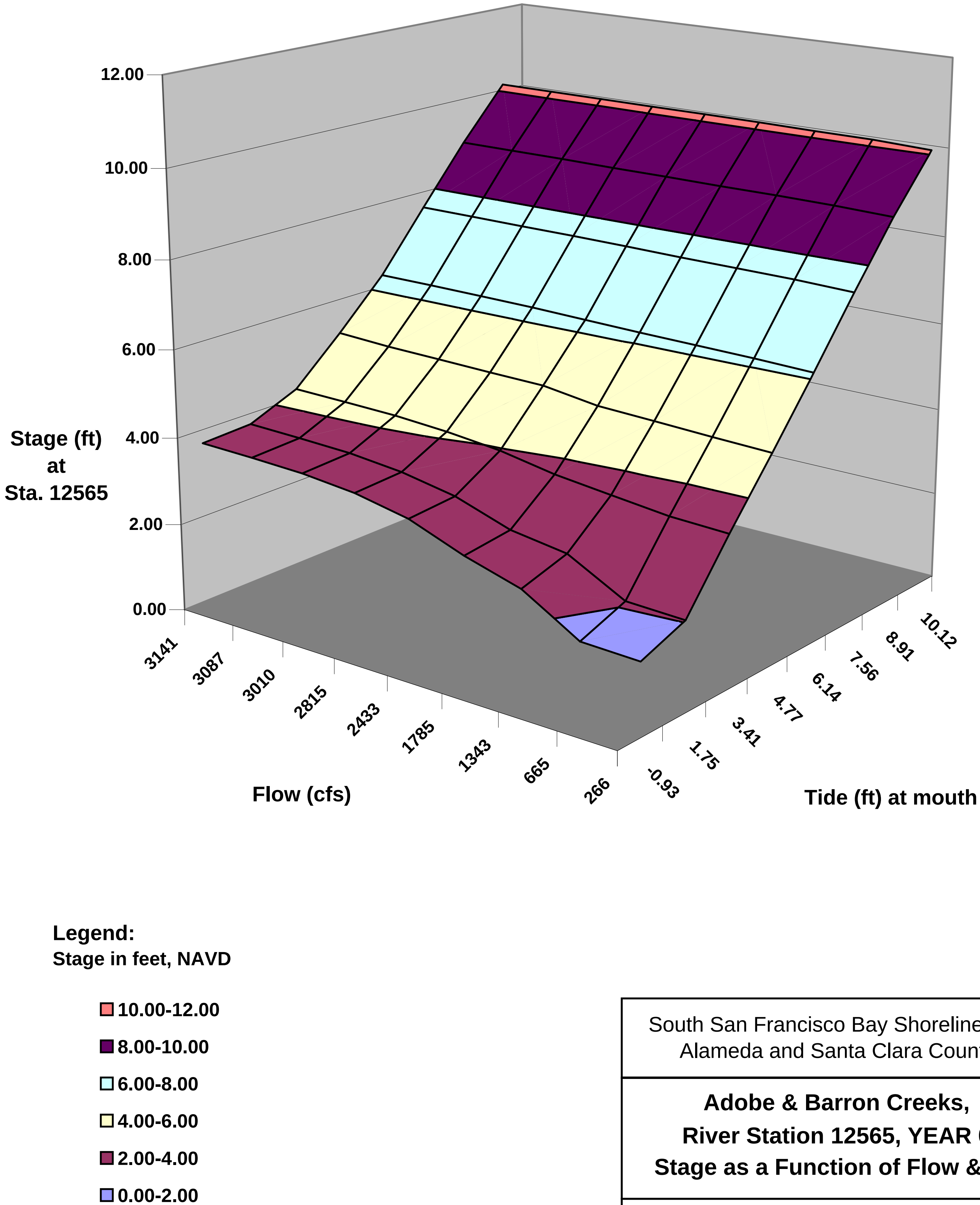
- **Year 50 Discretizations**
- **Year 50 Tides with 0.72 ft of Sea Level Rise**

South San Francisco Bay Shoreline Study Alameda and Santa Clara Counties
Permanente & Stevens Creeks, DS Boundary Conditions, Year 50 Tide Discretizations
US Army Corps of Engineers San Francisco District



South San Francisco Bay Shoreline Study Alameda and Santa Clara Counties
Guadalupe Slough & Sunnyvale West Channel, DS Boundary Conditions, Year 50 Tide Discretizations
US Army Corps of Engineers San Francisco District

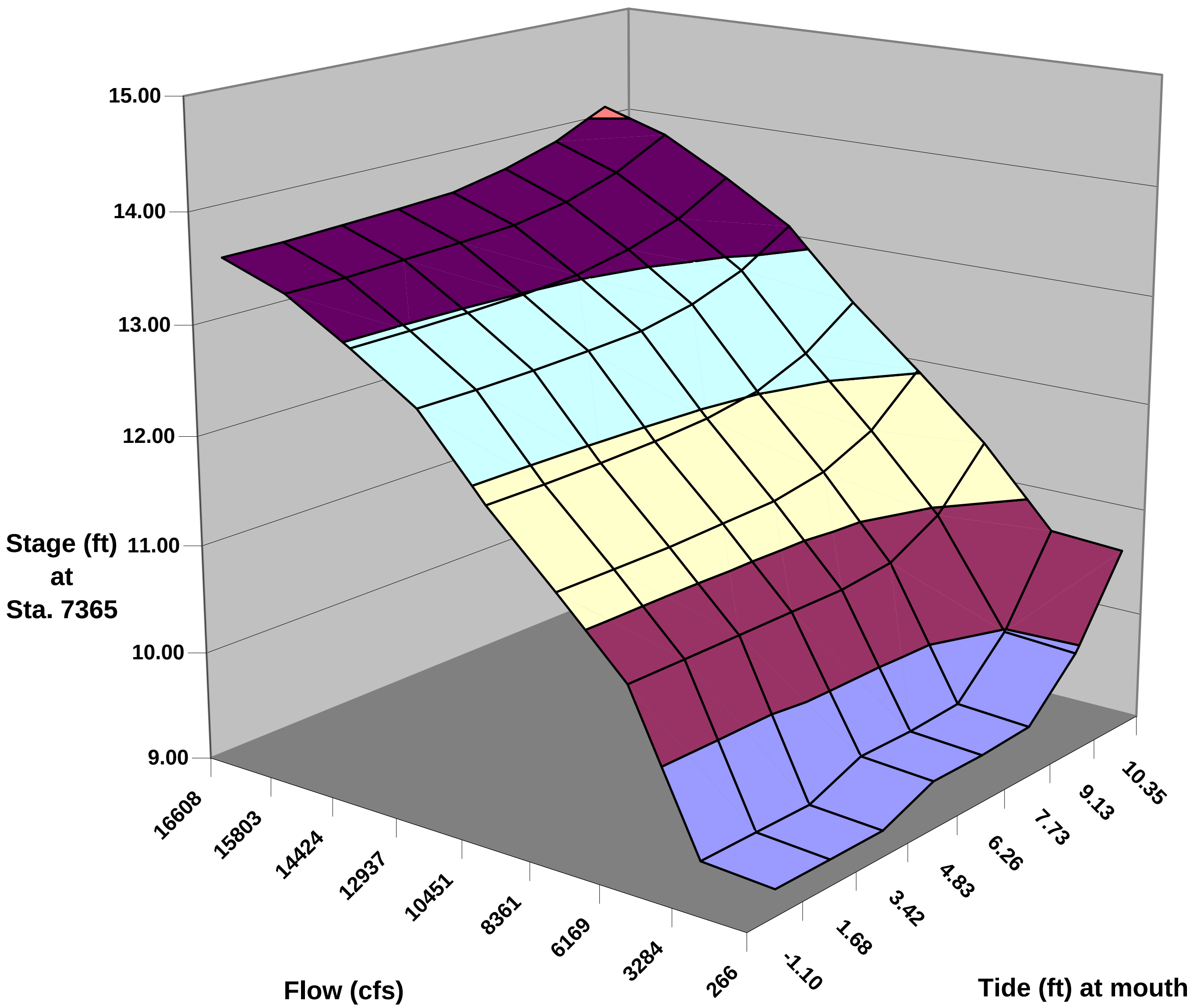

South San Francisco Bay
San Mateo County
Santa Clara County
Alameda County
Santa Clara County
Agua Caliente
Laguna Creek (E)
19248
Fremont Flood Control Channel
Mud Slough
2404
Coyote Creek
Coyote Creek Bypass
376
Lower Penitencia
7365
Alviso Slough
Guadalupe Slough
Milpitas
9983
12565
4896
8793.68
Matadero Creek
Barron Creek
Adobe Creek
Permanente Creek
Stevens Creek
24481.23
27774
10652
Guadalupe River
Coyote Creek
Calabazas Creek
Mountain View
Sunnyvale
Los Altos
Santa Clara
San Jose
Legend
CFA Index Points
Creek (Study Limits)
Highway
County Line
11983 River Station
Scale - 1:79,902
0 0.25 0.5 1 1.5 2
Miles
January 2010
PLATE 40
South San Francisco Bay Shoreline Study,
Santa Clara and Alameda County
Coincident Frequency Analysis
Index Points
US Army Corps of Engineers
San Francisco District



South San Francisco Bay Shoreline Study
Alameda and Santa Clara Counties

Adobe & Barron Creeks,
River Station 12565, YEAR 0
Stage as a Function of Flow & Tide

US Army Corps of Engineers
San Francisco District



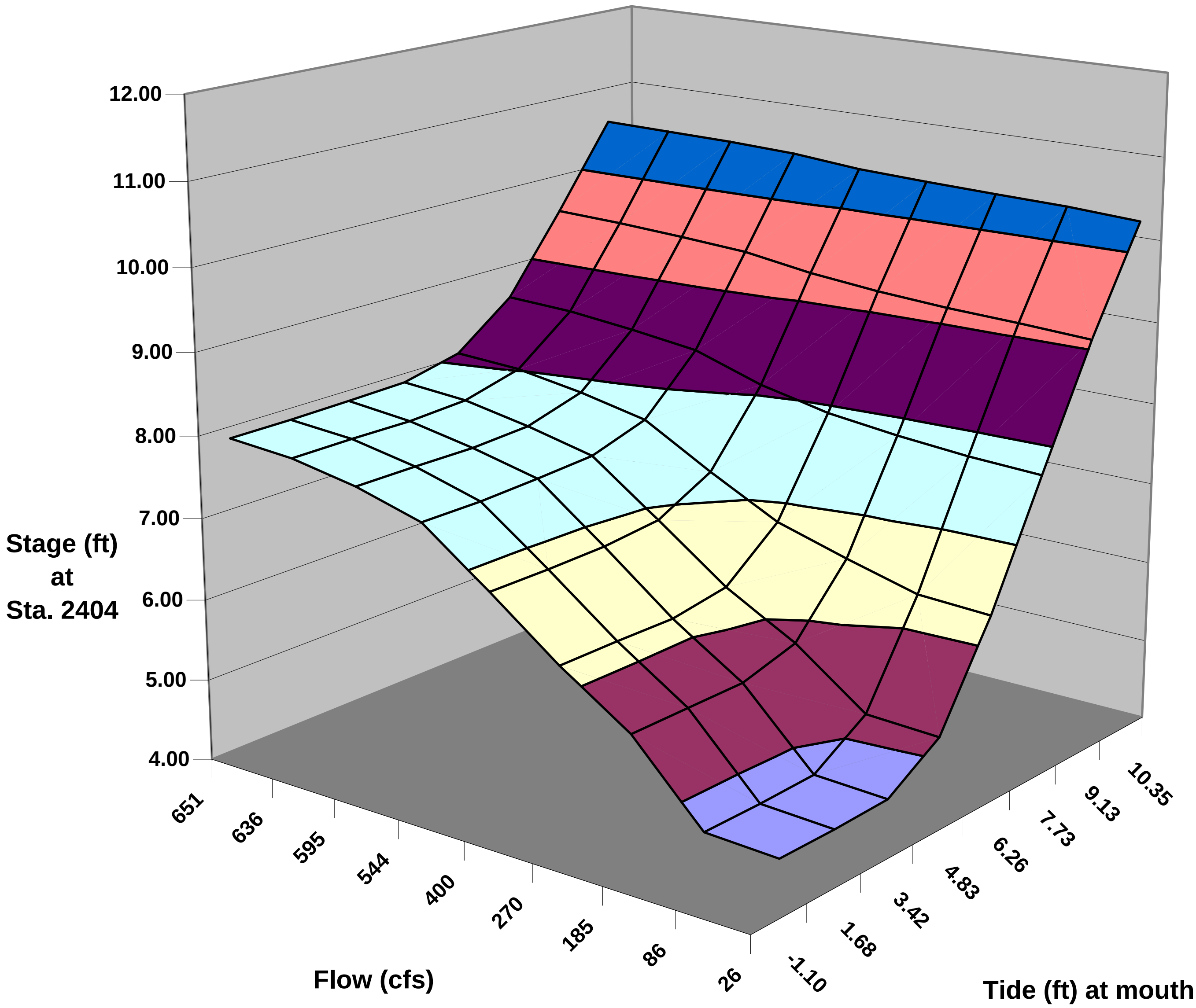
Legend:
Stage in feet, NAVD

- **14.00-15.00**
- **13.00-14.00**
- **12.00-13.00**
- **11.00-12.00**
- **10.00-11.00**
- **9.00-10.00**

South San Francisco Bay Shoreline Study
Alameda and Santa Clara Counties

Coyote Creek,
River Station 7365, YEAR 0
Stage as a Function of Flow & Tide

US Army Corps of Engineers
San Francisco District



Legend:
Stage in feet, NAVD

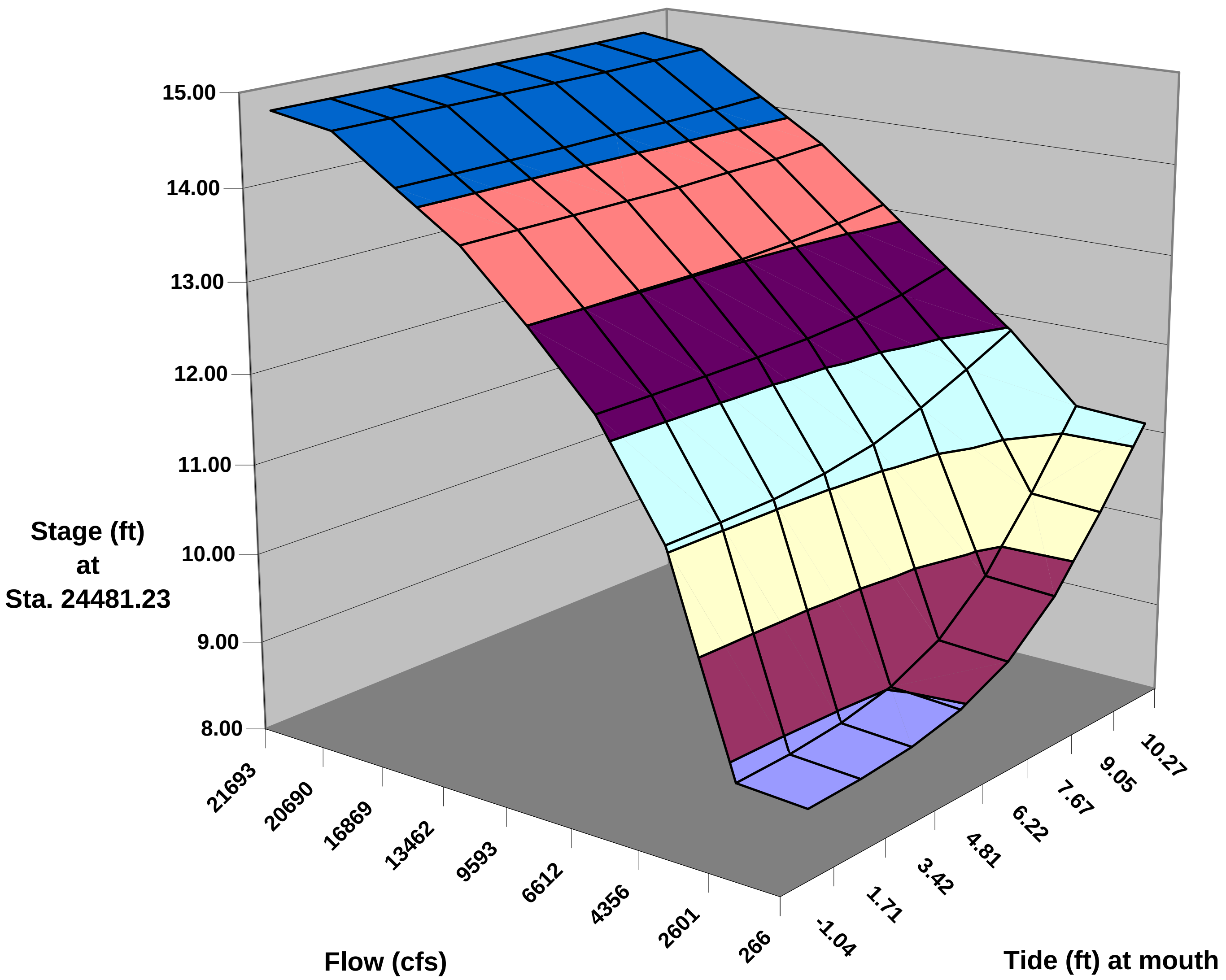
- 11.00-12.00
- 10.00-11.00
- 9.00-10.00
- 8.00-9.00
- 7.00-8.00
- 6.00-7.00
- 5.00-6.00
- 4.00-5.00

South San Francisco Bay Shoreline Study
Alameda and Santa Clara Counties

Fremont Flood Control Channel,
River Station 2404, YEAR 0
Stage as a Function of Flow & Tide

US Army Corps of Engineers
San Francisco District

PLATE 41c



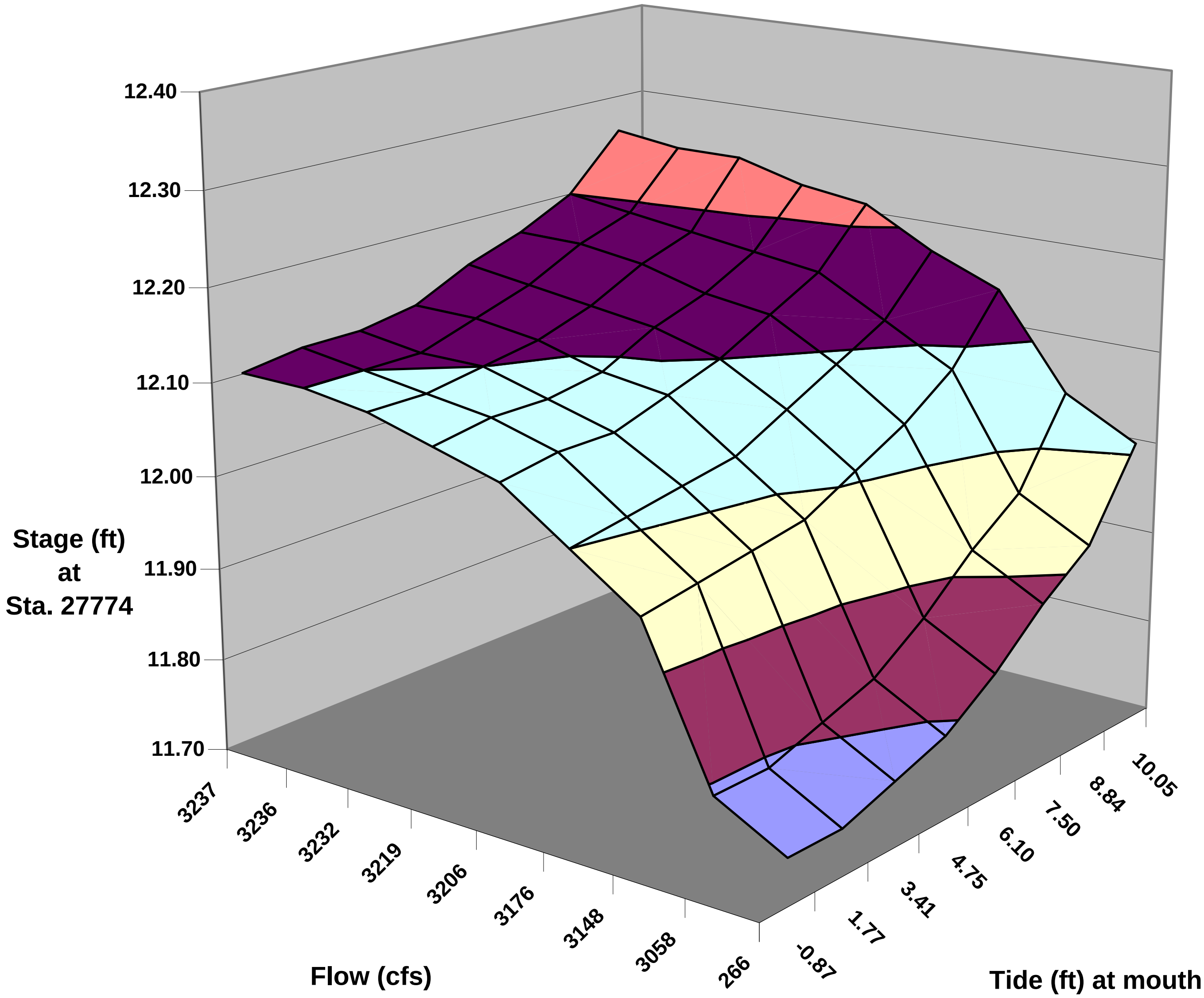
Legend:
Stage in feet, NAVD

- **14.00-15.00**
- **13.00-14.00**
- **12.00-13.00**
- **11.00-12.00**
- **10.00-11.00**
- **9.00-10.00**
- **8.00-9.00**

South San Francisco Bay Shoreline Study
Alameda and Santa Clara Counties

Guadalupe River,
River Station 24481.23, YEAR 0
Stage as a Function of Flow & Tide

US Army Corps of Engineers
San Francisco District



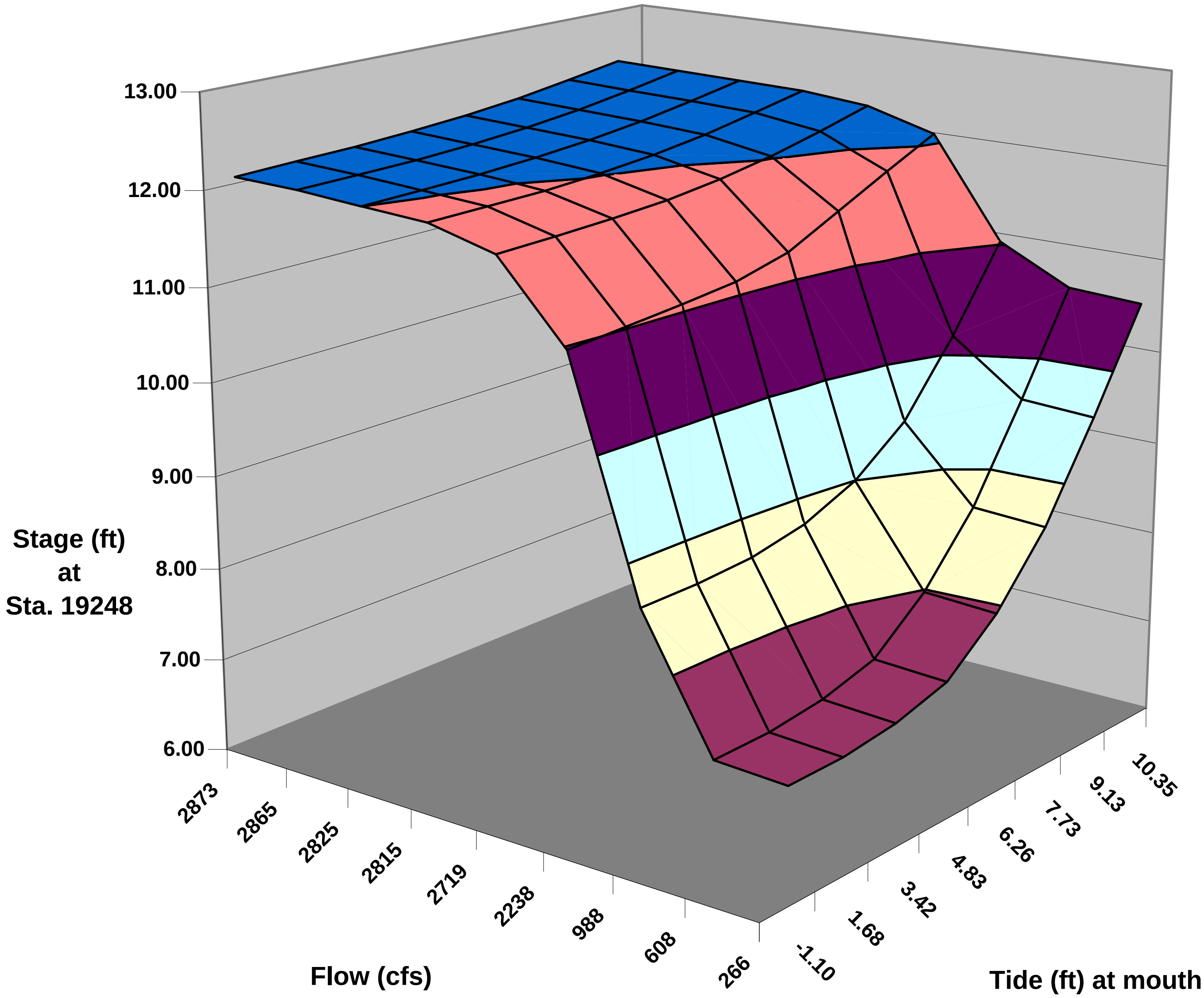
Legend:
Stage in feet, NAVD

- **12.30-12.40**
- **12.20-12.30**
- **12.10-12.20**
- **12.00-12.10**
- **11.90-12.00**
- **11.80-11.90**
- **11.70-11.80**

South San Francisco Bay Shoreline Study
Alameda and Santa Clara Counties

Guadalupe Slough,
River Station 27774, YEAR 0
Stage as a Function of Flow & Tide

US Army Corps of Engineers
San Francisco District



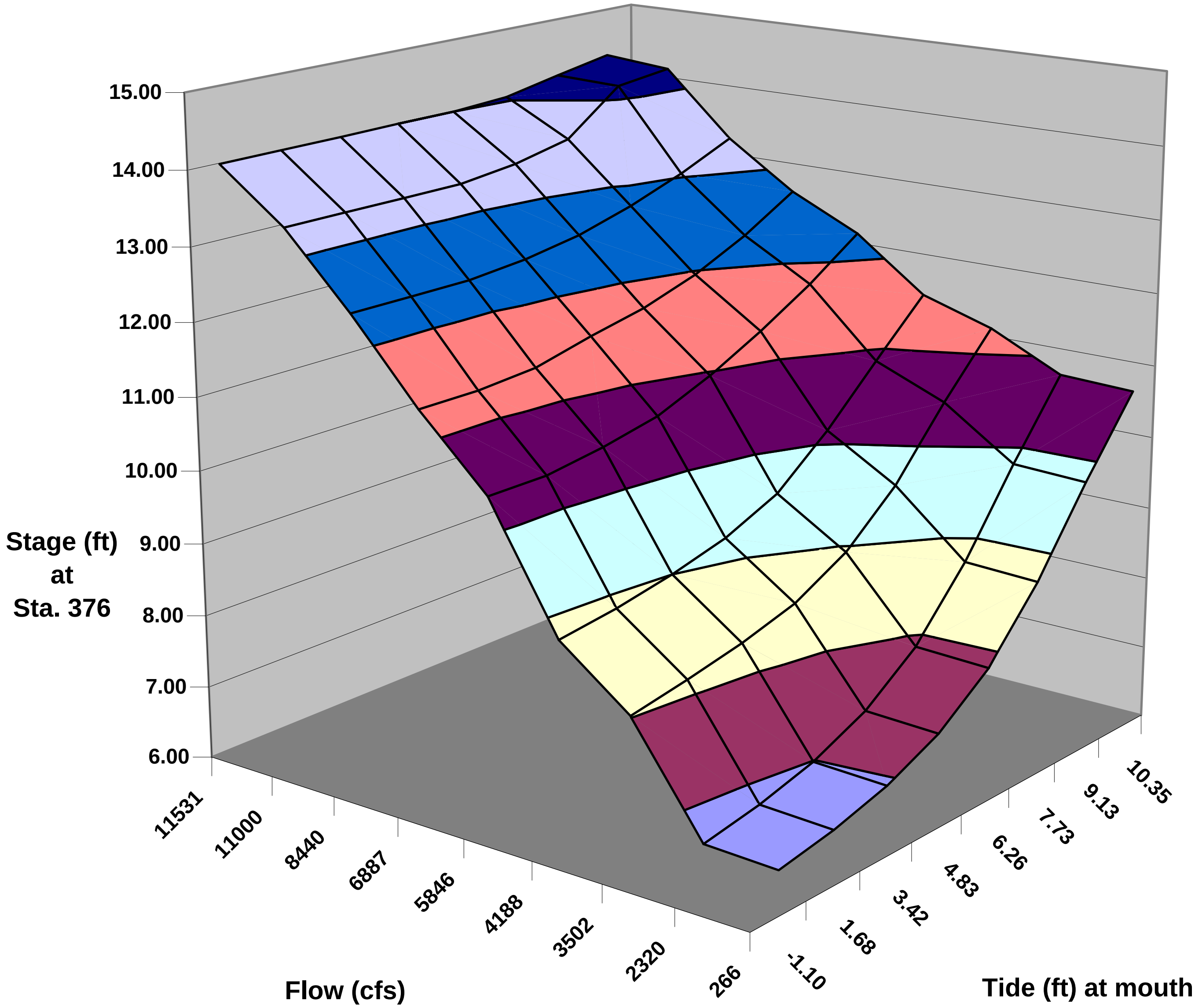
Legend:
Stage in feet, NAVD

- **12.00-13.00**
- **11.00-12.00**
- **10.00-11.00**
- **9.00-10.00**
- **8.00-9.00**
- **7.00-8.00**
- **6.00-7.00**

South San Francisco Bay Shoreline Study
Alameda and Santa Clara Counties

Laguna Creek,
River Station 19248, YEAR 0
Stage as a Function of Flow & Tide

US Army Corps of Engineers
San Francisco District



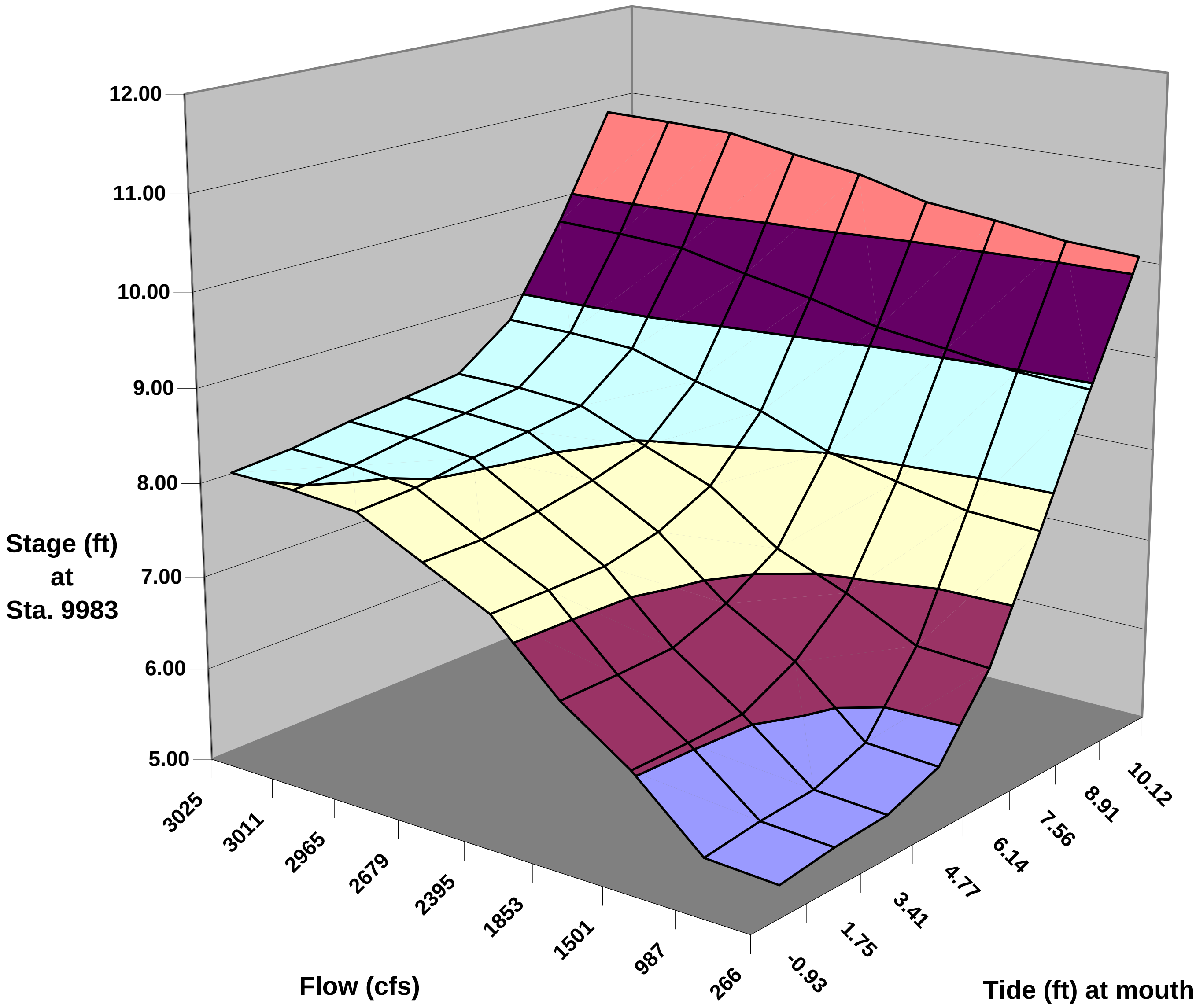
Legend:
Stage in feet, NAVD

- **14.00-15.00**
- **13.00-14.00**
- **12.00-13.00**
- **11.00-12.00**
- **10.00-11.00**
- **9.00-10.00**
- **8.00-9.00**
- **7.00-8.00**
- **6.00-7.00**

South San Francisco Bay Shoreline Study
Alameda and Santa Clara Counties

Lower Penitencia Creek,
River Station 376, YEAR 0
Stage as a Function of Flow & Tide

US Army Corps of Engineers
San Francisco District



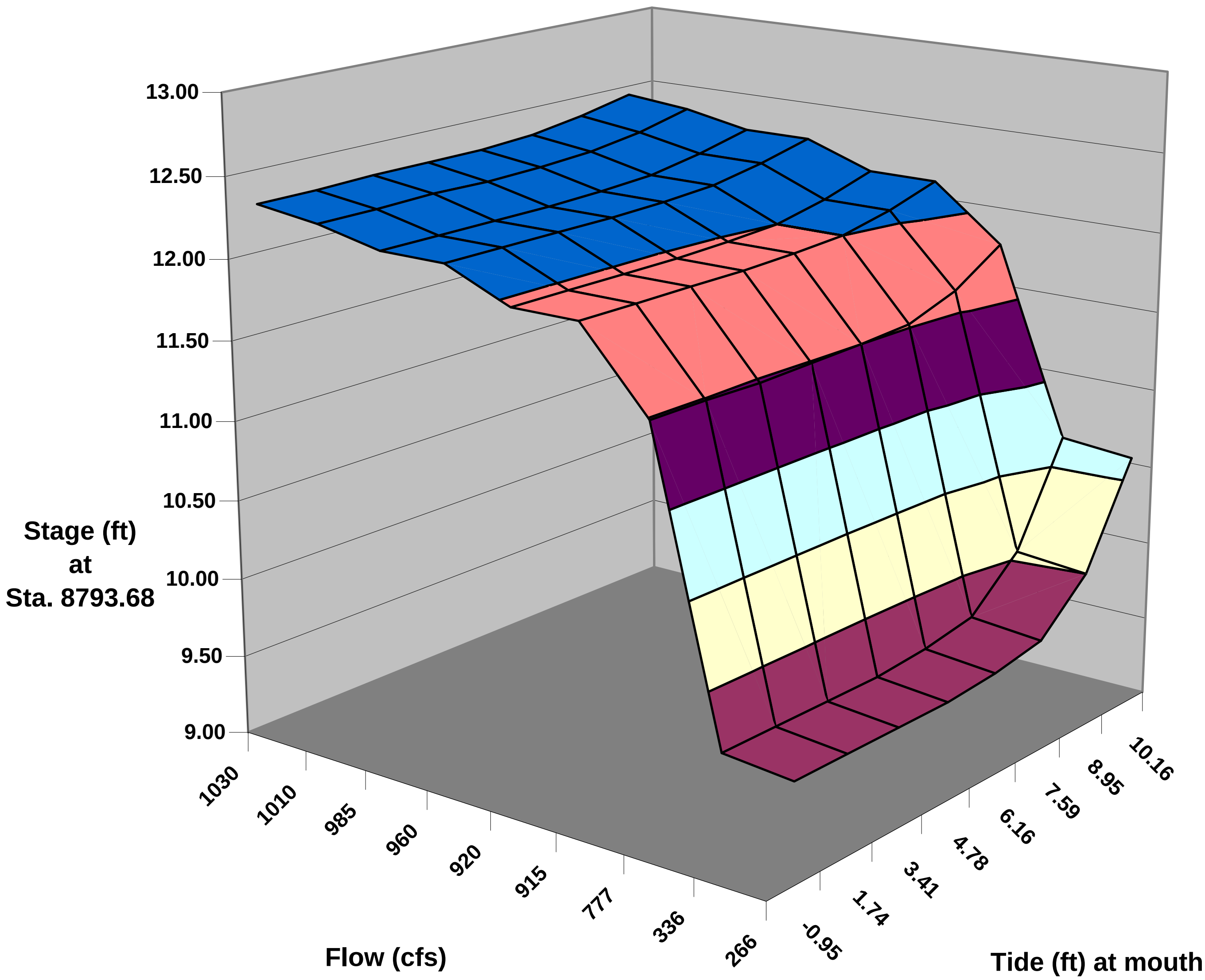
Legend:
Stage in feet, NAVD

- 11.00-12.00
- 10.00-11.00
- 9.00-10.00
- 8.00-9.00
- 7.00-8.00
- 6.00-7.00
- 5.00-6.00

South San Francisco Bay Shoreline Study
Alameda and Santa Clara Counties

Matadero Creek,
River Station 9983, YEAR 0
Stage as a Function of Flow & Tide

US Army Corps of Engineers
San Francisco District



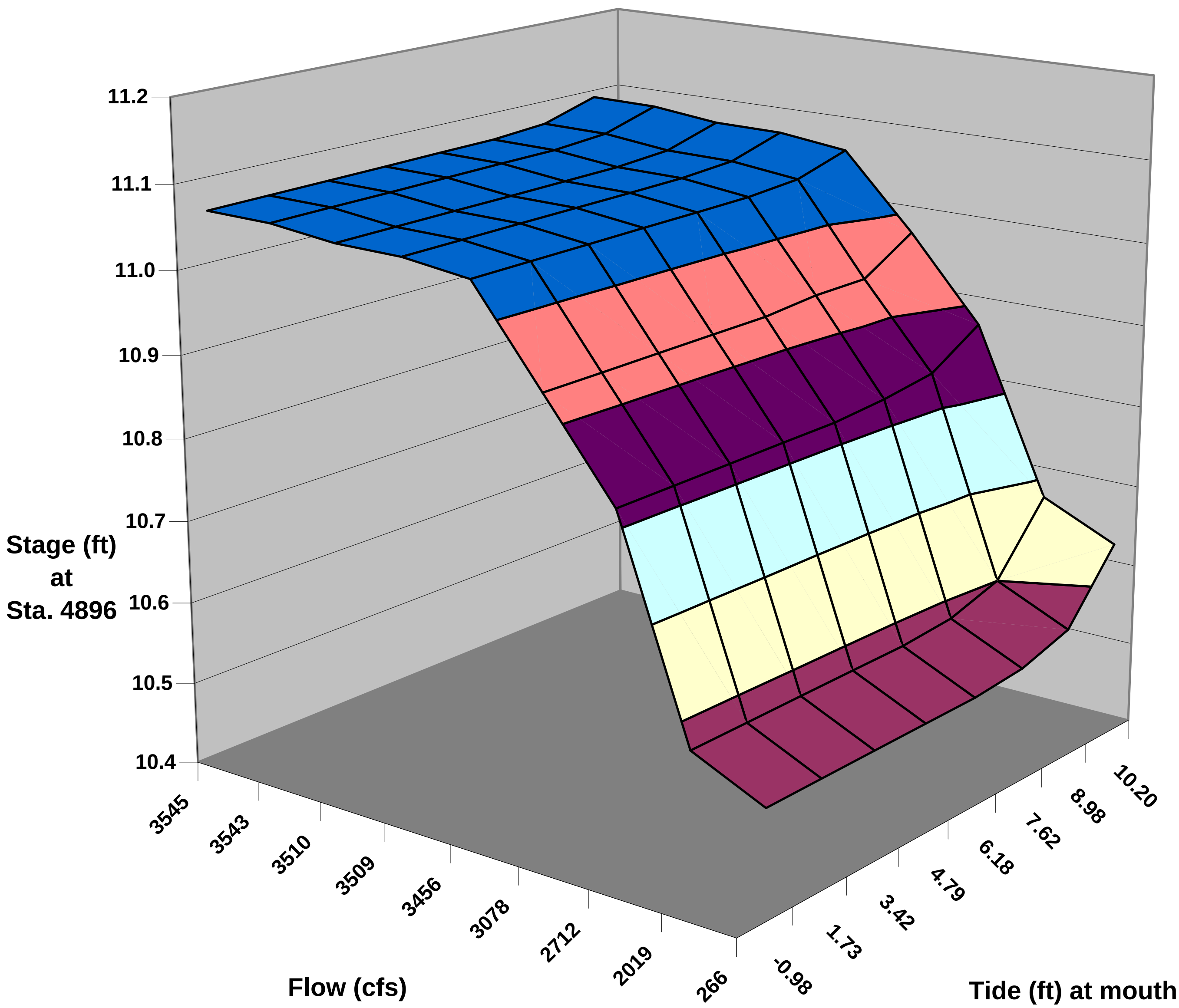
Legend:
Stage in feet, NAVD

- 12.50-13.00
- 12.00-12.50
- 11.50-12.00
- 11.00-11.50
- 10.50-11.00
- 10.00-10.50
- 9.50-10.00
- 9.00-9.50

South San Francisco Bay Shoreline Study
Alameda and Santa Clara Counties

Permanente Creek,
River Station 8793.68, YEAR 0
Stage as a Function of Flow & Tide

US Army Corps of Engineers
San Francisco District



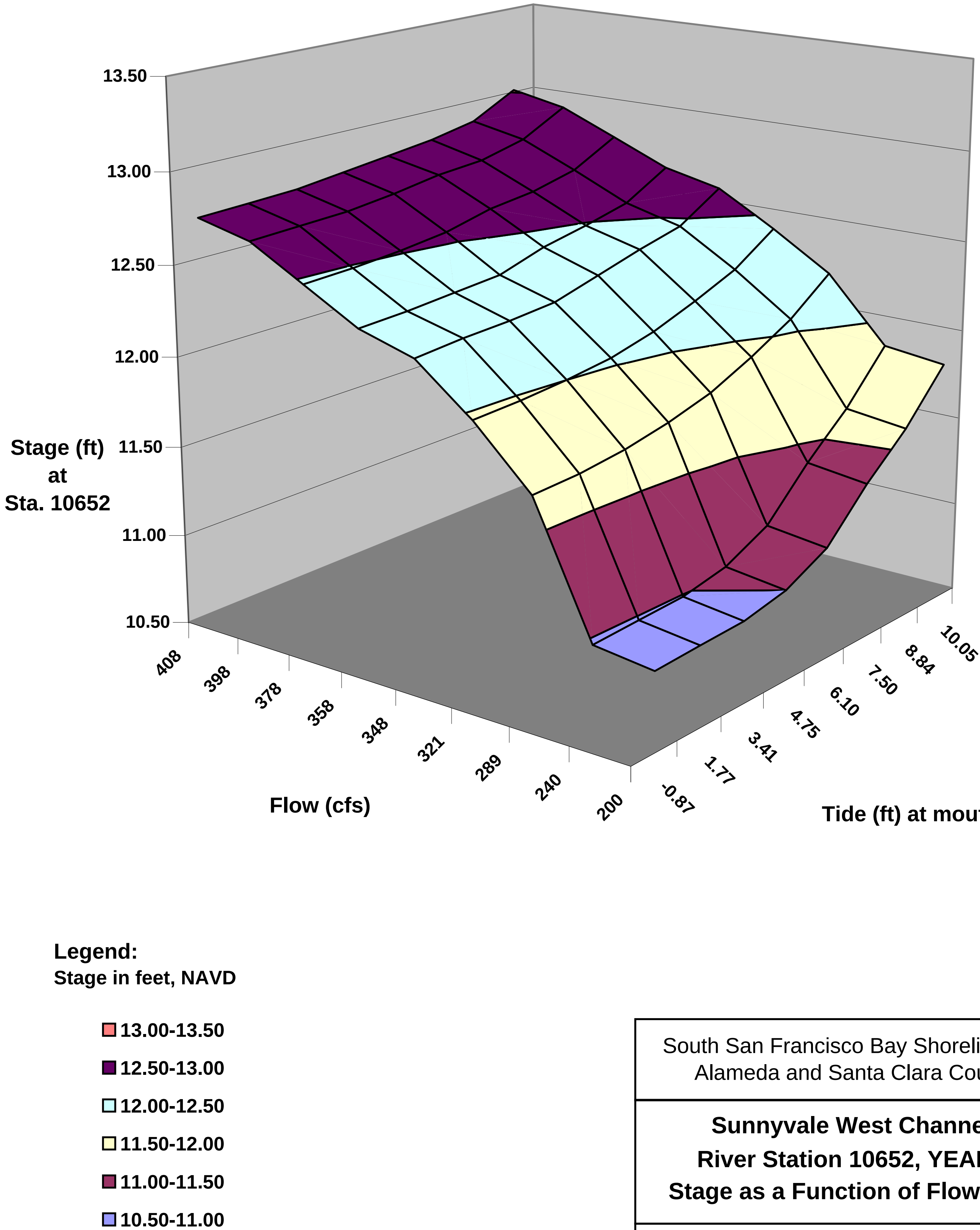
Legend:
Stage in feet, NAVD

- **11.100-11.200**
- **11.000-11.100**
- **10.900-11.000**
- **10.800-10.900**
- **10.700-10.800**
- **10.600-10.700**
- **10.500-10.600**
- **10.400-10.500**

South San Francisco Bay Shoreline Study
Alameda and Santa Clara Counties

Stevens Creek,
River Station 4896, YEAR 0
Stage as a Function of Flow & Tide

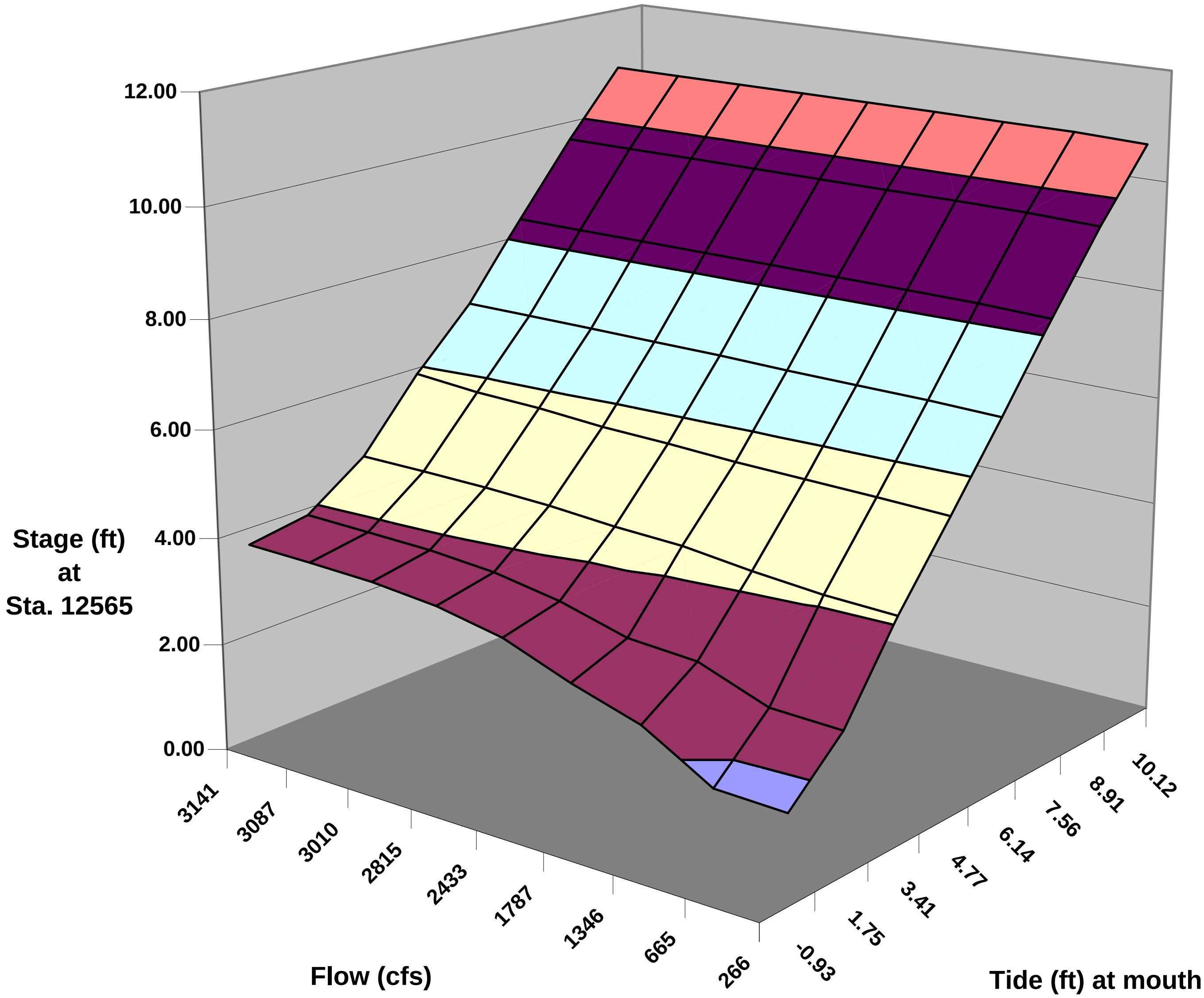
US Army Corps of Engineers
San Francisco District



South San Francisco Bay Shoreline Study
Alameda and Santa Clara Counties

Sunnyvale West Channel,
River Station 10652, YEAR 0
Stage as a Function of Flow & Tide

US Army Corps of Engineers
San Francisco District



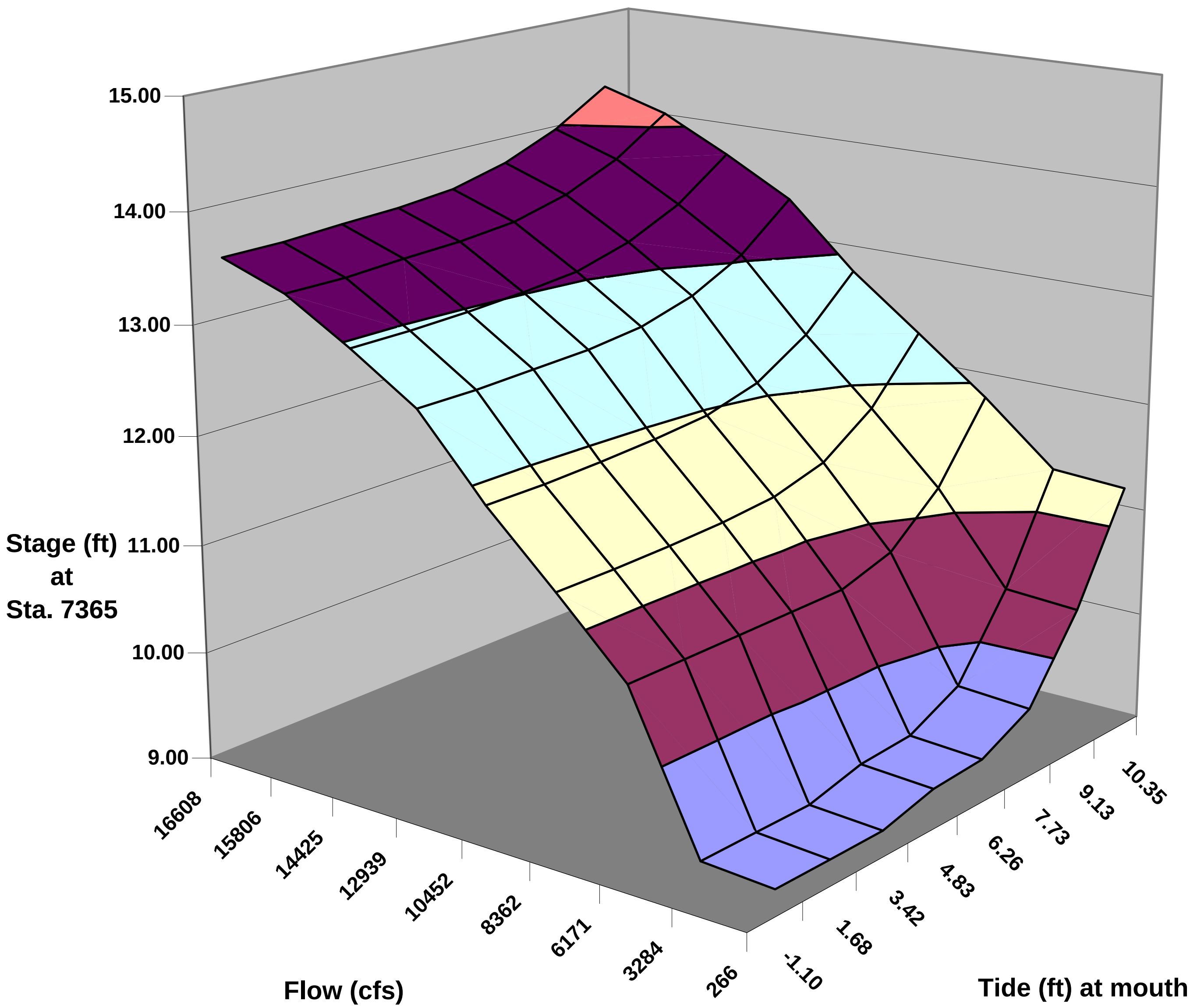
Legend:
Stage in feet, NAVD

- **10.00-12.00**
- **8.00-10.00**
- **6.00-8.00**
- **4.00-6.00**
- **2.00-4.00**
- **0.00-2.00**

South San Francisco Bay Shoreline Study
Alameda and Santa Clara Counties

Adobe & Barron Creeks,
River Station 12565, YEAR 50
Stage as a Function of Flow & Tide

US Army Corps of Engineers
San Francisco District



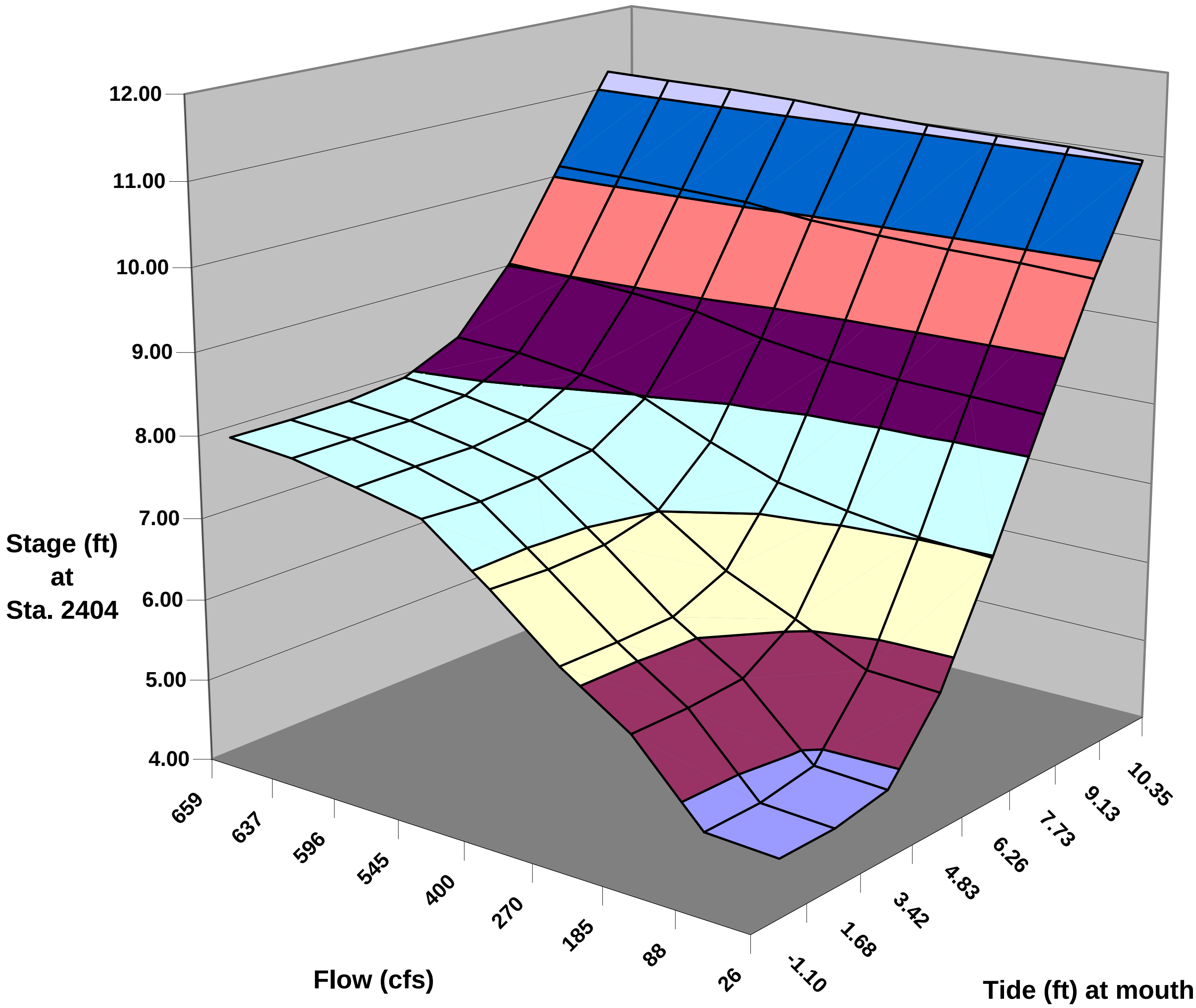
Legend:
Stage in feet, NAVD

- **14.00-15.00**
- **13.00-14.00**
- **12.00-13.00**
- **11.00-12.00**
- **10.00-11.00**
- **9.00-10.00**

South San Francisco Bay Shoreline Study
Alameda and Santa Clara Counties

Coyote Creek,
River Station 7365, YEAR 50
Stage as a Function of Flow & Tide

US Army Corps of Engineers
San Francisco District



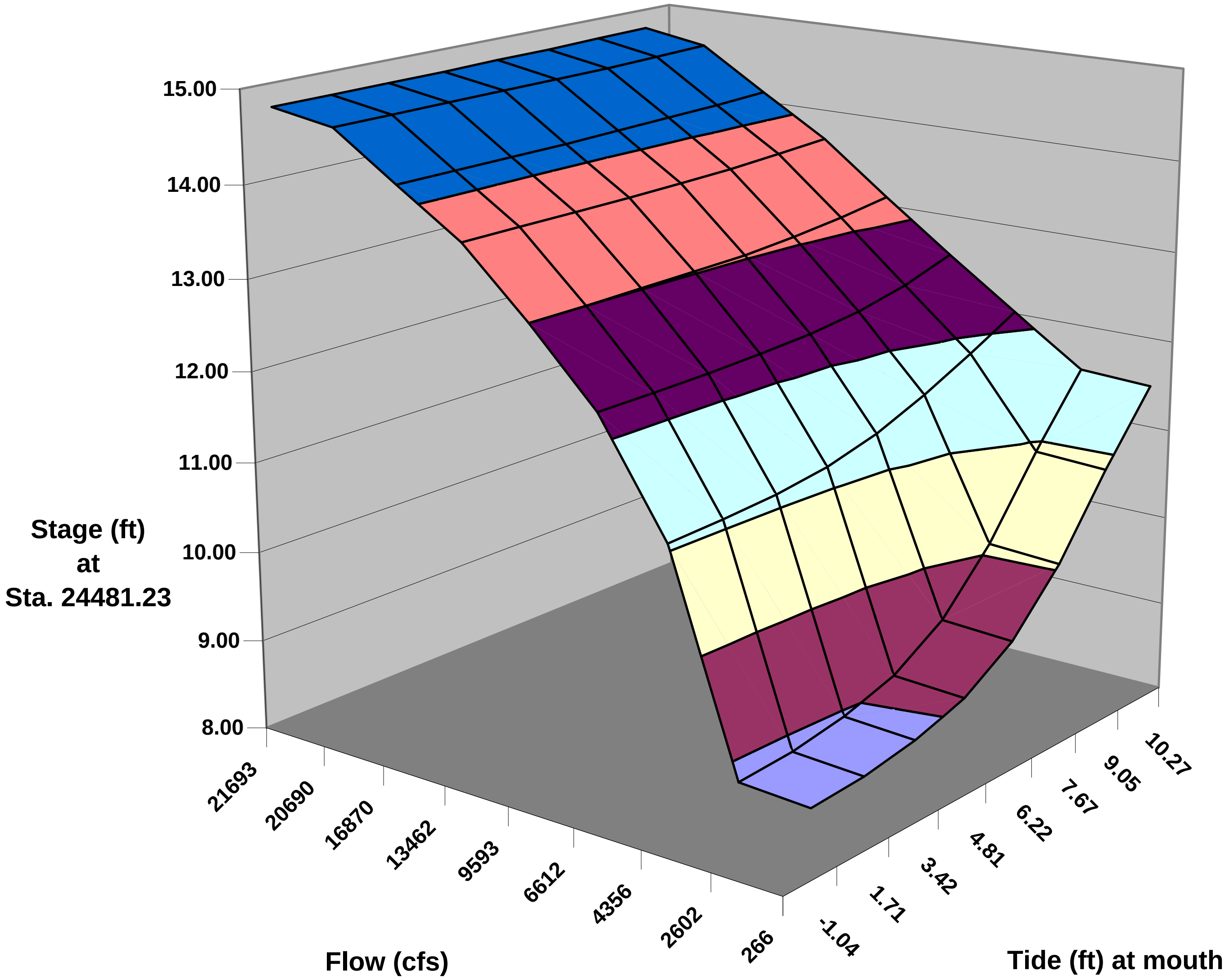
Legend:
Stage in feet, NAVD

- 11.00-12.00
- 10.00-11.00
- 9.00-10.00
- 8.00-9.00
- 7.00-8.00
- 6.00-7.00
- 5.00-6.00
- 4.00-5.00

South San Francisco Bay Shoreline Study
Alameda and Santa Clara Counties

Fremont Flood Control Channel,
River Station 2404, YEAR 50
Stage as a Function of Flow & Tide

US Army Corps of Engineers
San Francisco District



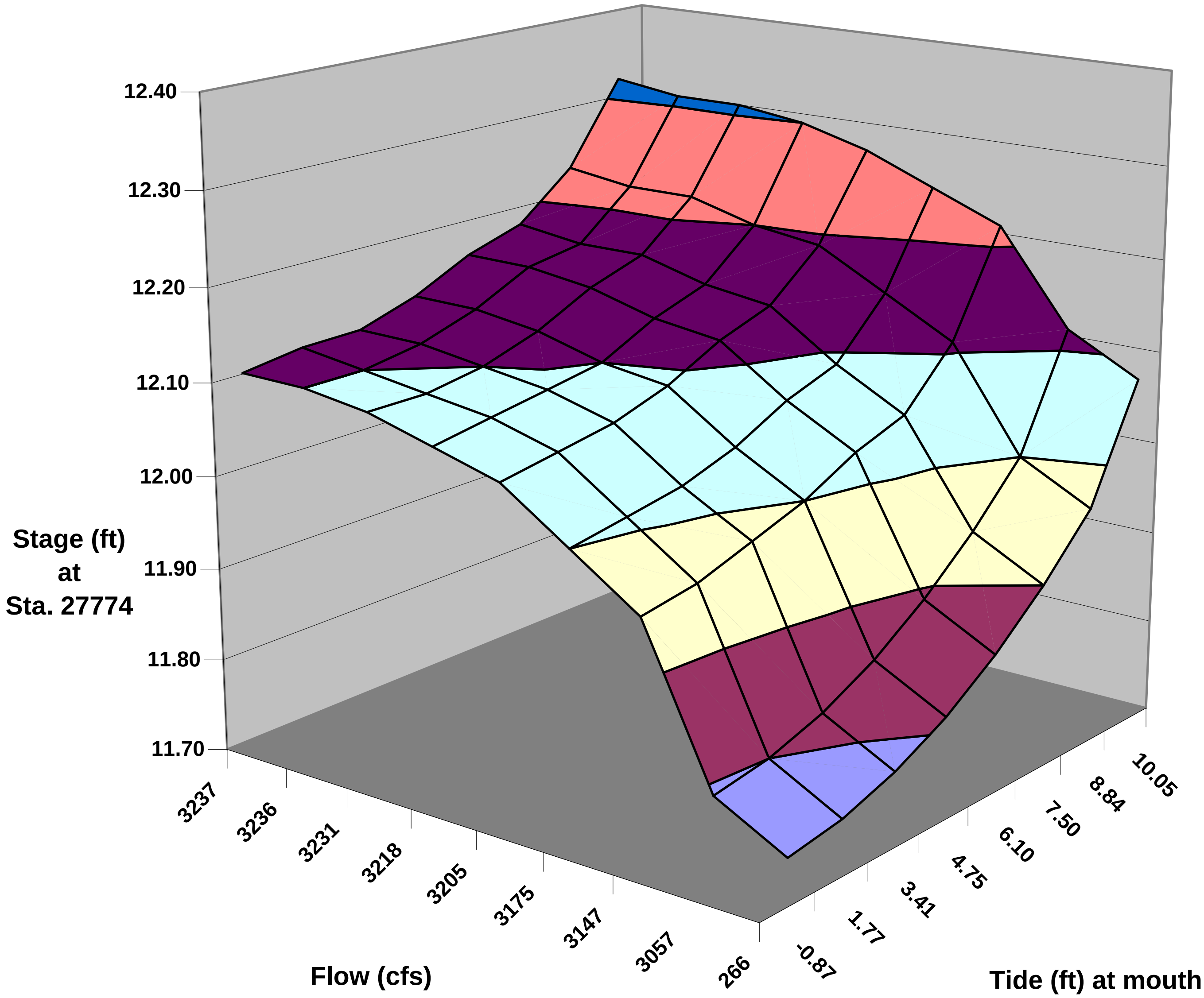
Legend:
Stage in feet, NAVD

- **14.00-15.00**
- **13.00-14.00**
- **12.00-13.00**
- **11.00-12.00**
- **10.00-11.00**
- **9.00-10.00**
- **8.00-9.00**

South San Francisco Bay Shoreline Study
Alameda and Santa Clara Counties

Guadalupe River,
River Station 24481.23, YEAR 50
Stage as a Function of Flow & Tide

US Army Corps of Engineers
San Francisco District



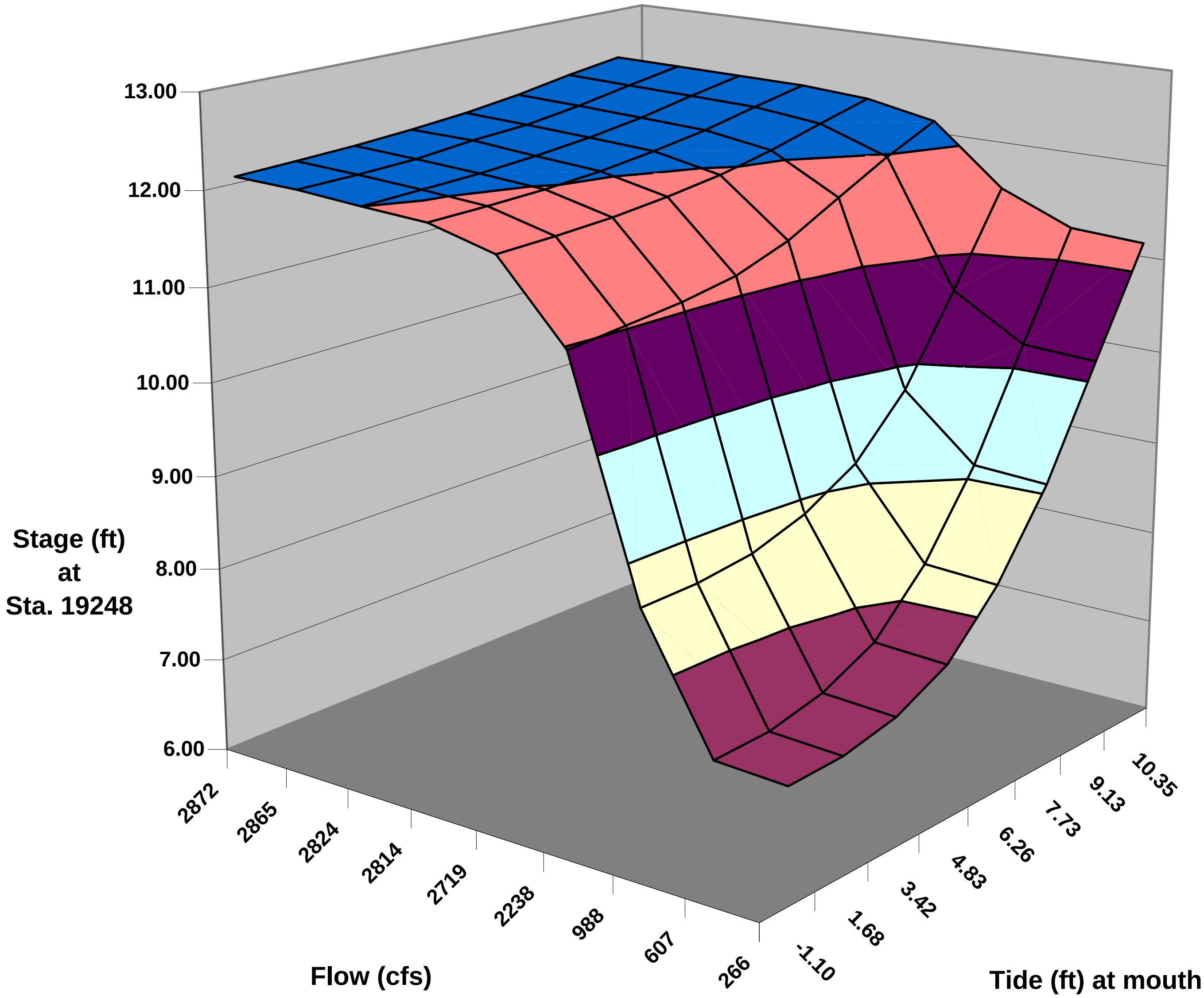
Legend:
Stage in feet, NAVD

- **12.30-12.40**
- **12.20-12.30**
- **12.10-12.20**
- **12.00-12.10**
- **11.90-12.00**
- **11.80-11.90**
- **11.70-11.80**

South San Francisco Bay Shoreline Study
Alameda and Santa Clara Counties

Guadalupe Slough,
River Station 27774, YEAR 50
Stage as a Function of Flow & Tide

US Army Corps of Engineers
San Francisco District



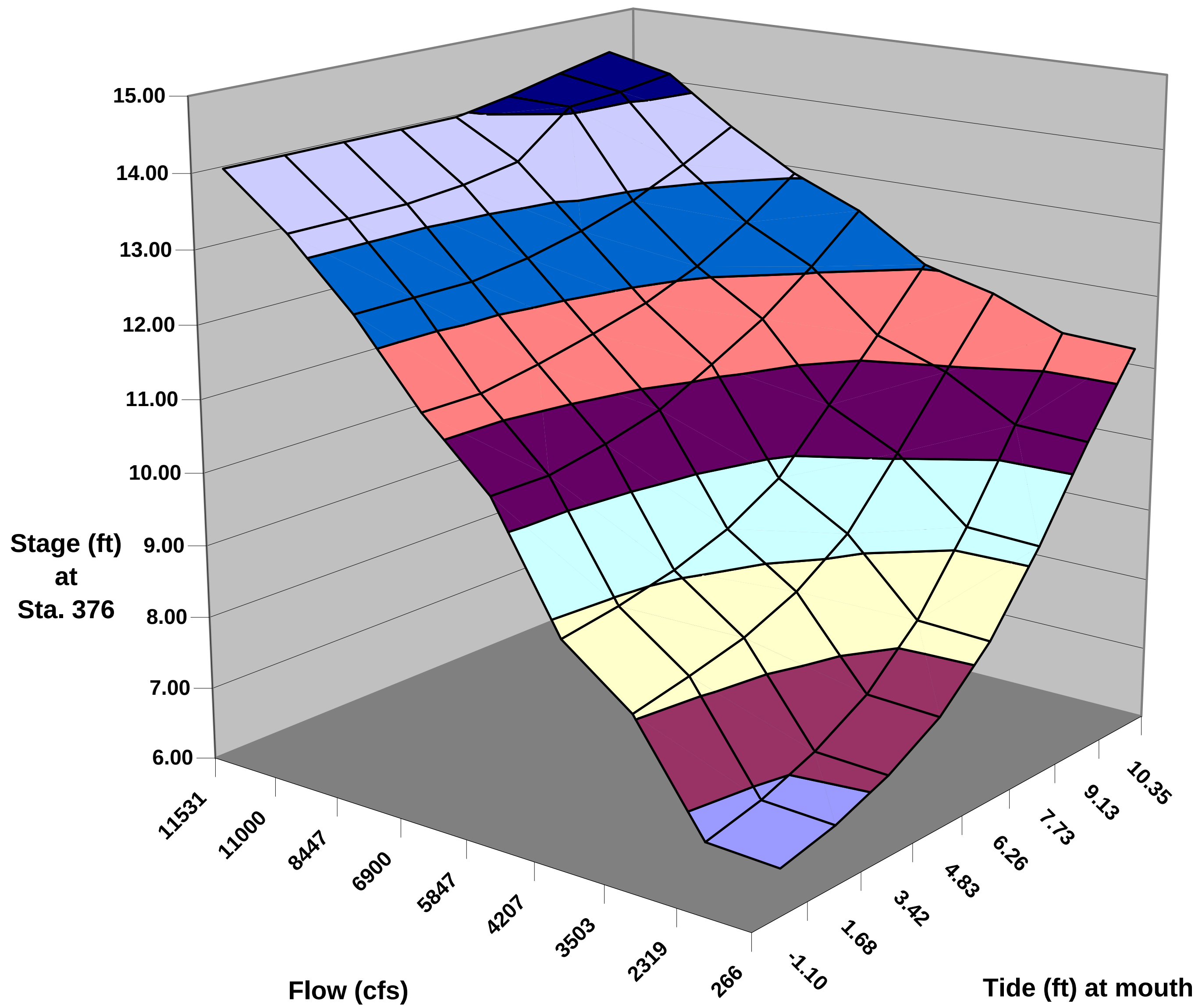
Legend:
Stage in feet, NAVD

- **12.00-13.00**
- **11.00-12.00**
- **10.00-11.00**
- **9.00-10.00**
- **8.00-9.00**
- **7.00-8.00**
- **6.00-7.00**

South San Francisco Bay Shoreline Study
Alameda and Santa Clara Counties

Laguna Creek,
River Station 19248, YEAR 50
Stage as a Function of Flow & Tide

US Army Corps of Engineers
San Francisco District



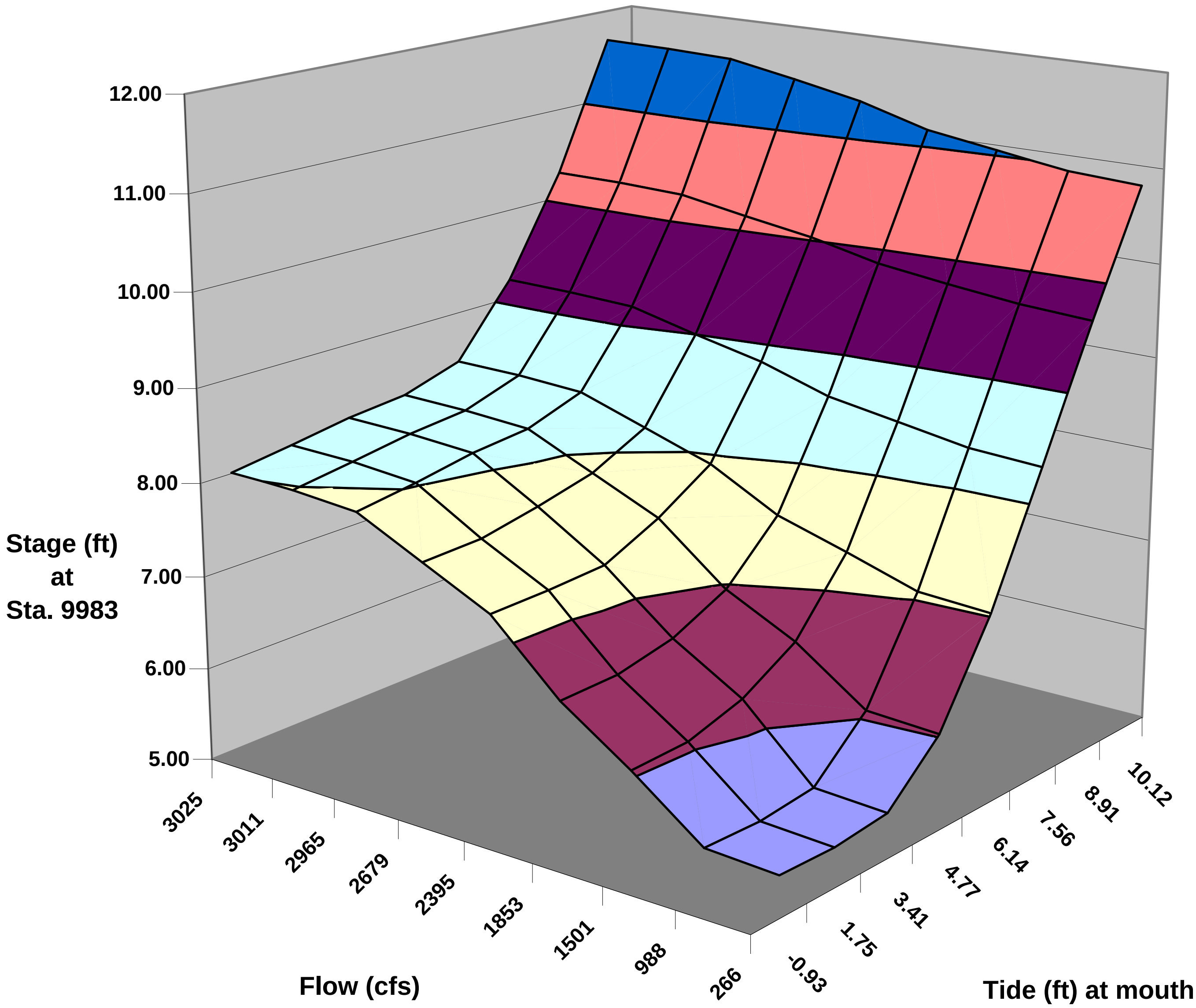
Legend:
Stage in feet, NAVD

- 14.00-15.00
- 13.00-14.00
- 12.00-13.00
- 11.00-12.00
- 10.00-11.00
- 9.00-10.00
- 8.00-9.00
- 7.00-8.00
- 6.00-7.00

South San Francisco Bay Shoreline Study
Alameda and Santa Clara Counties

Lower Penitencia Creek,
River Station 376, YEAR 50
Stage as a Function of Flow & Tide

US Army Corps of Engineers
San Francisco District



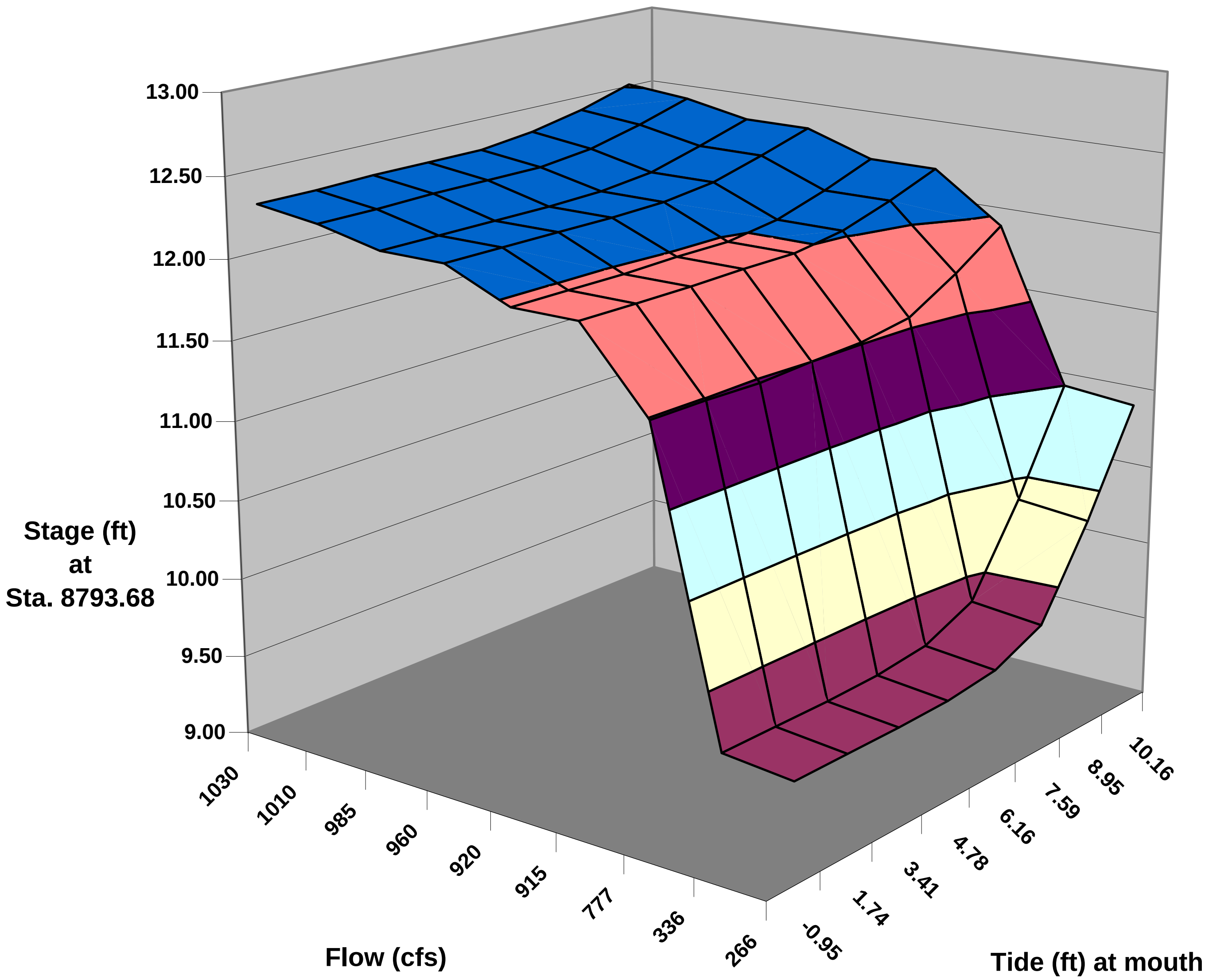
Legend:
Stage in feet, NAVD

- **11.00-12.00**
- **10.00-11.00**
- **9.00-10.00**
- **8.00-9.00**
- **7.00-8.00**
- **6.00-7.00**
- **5.00-6.00**

South San Francisco Bay Shoreline Study
Alameda and Santa Clara Counties

Matadero Creek,
River Station 9983, YEAR 50
Stage as a Function of Flow & Tide

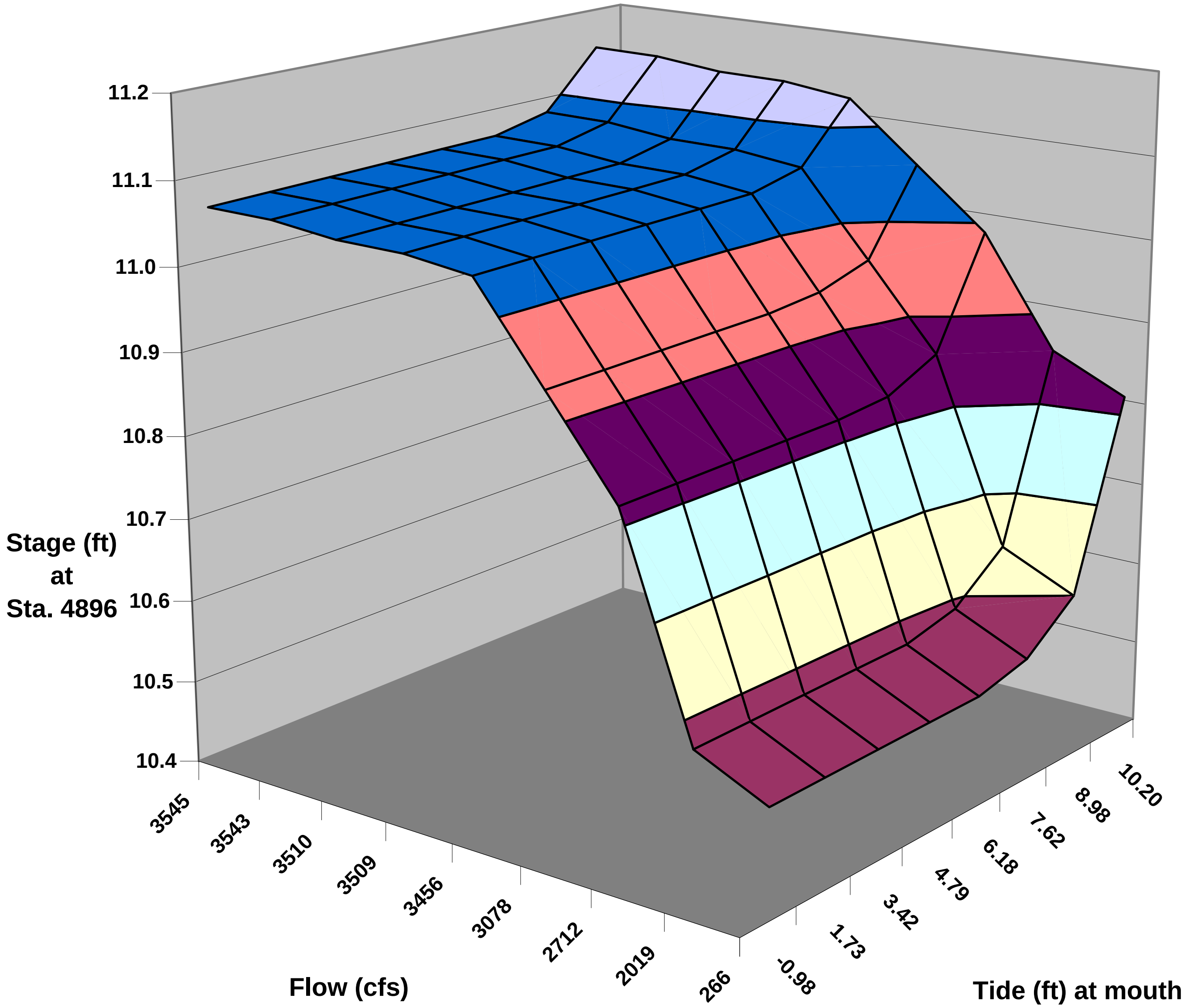
US Army Corps of Engineers
San Francisco District



Legend:
Stage in feet, NAVD

- 12.50-13.00
- 12.00-12.50
- 11.50-12.00
- 11.00-11.50
- 10.50-11.00
- 10.00-10.50
- 9.50-10.00
- 9.00-9.50

South San Francisco Bay Shoreline Study Alameda and Santa Clara Counties
Permanente Creek, River Station 8793.68, YEAR 50 Stage as a Function of Flow & Tide
US Army Corps of Engineers San Francisco District



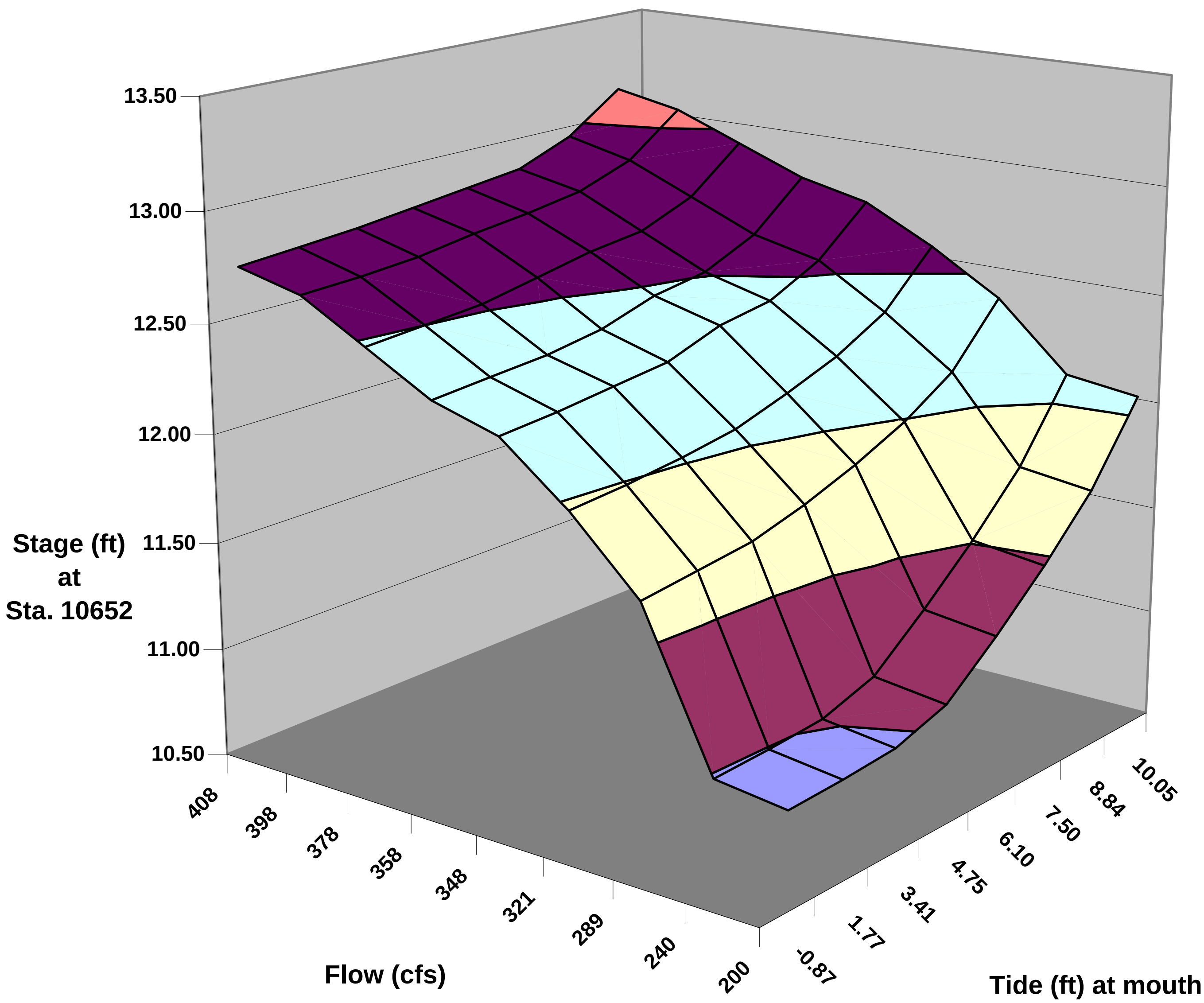
Legend:
Stage in feet, NAVD

- **11.10-11.20**
- **11.00-11.10**
- **10.90-11.00**
- **10.80-10.90**
- **10.70-10.80**
- **10.60-10.70**
- **10.50-10.60**
- **10.40-10.50**

South San Francisco Bay Shoreline Study
Alameda and Santa Clara Counties

Stevens Creek,
River Station 4896, YEAR 50
Stage as a Function of Flow & Tide

US Army Corps of Engineers
San Francisco District



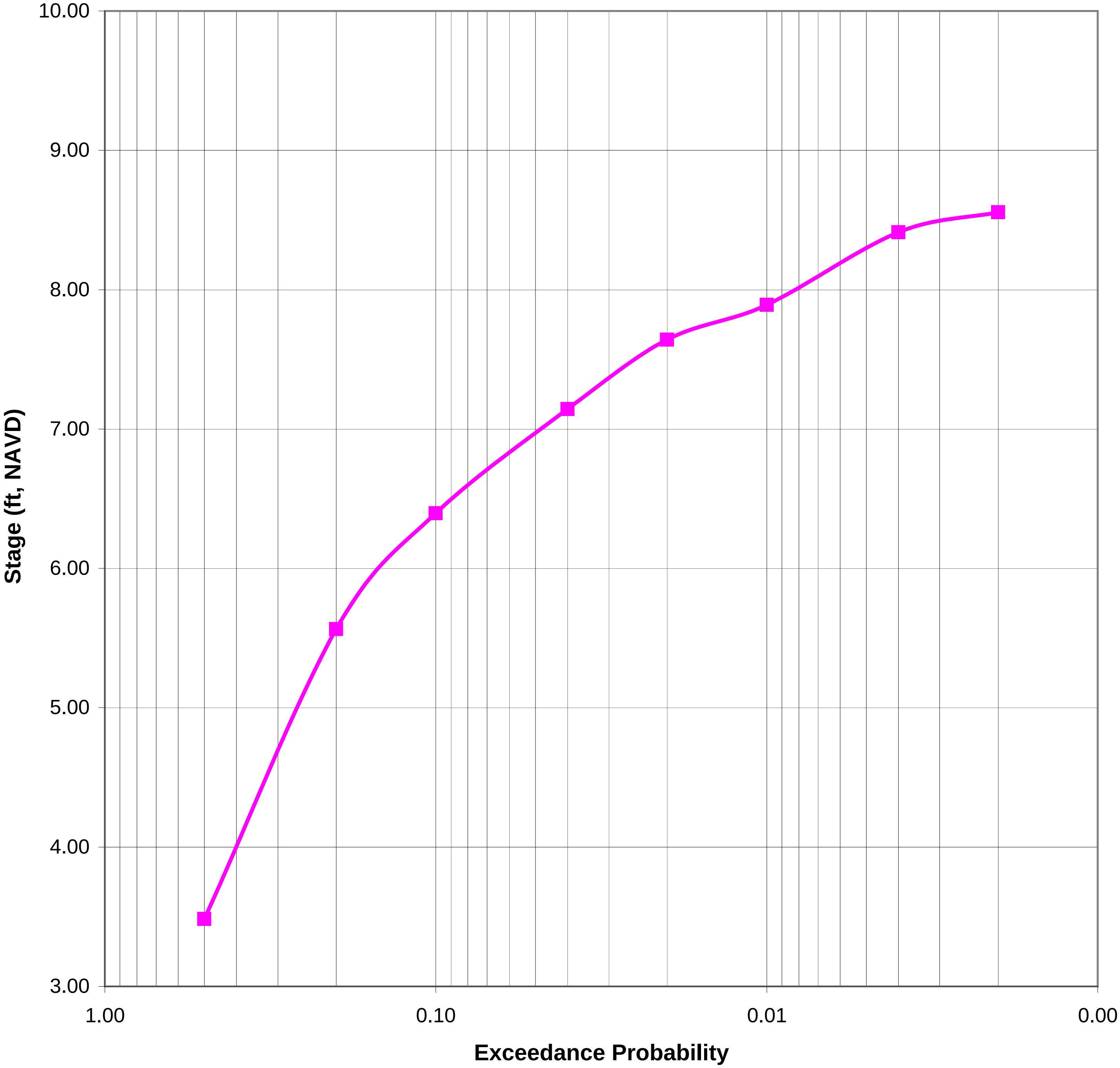
Legend:
Stage in feet, NAVD

- **13.00-13.50**
- **12.50-13.00**
- **12.00-12.50**
- **11.50-12.00**
- **11.00-11.50**
- **10.50-11.00**

South San Francisco Bay Shoreline Study
Alameda and Santa Clara Counties

Sunnyvale West Channel,
River Station 10652, YEAR 50
Stage as a Function of Flow & Tide

US Army Corps of Engineers
San Francisco District



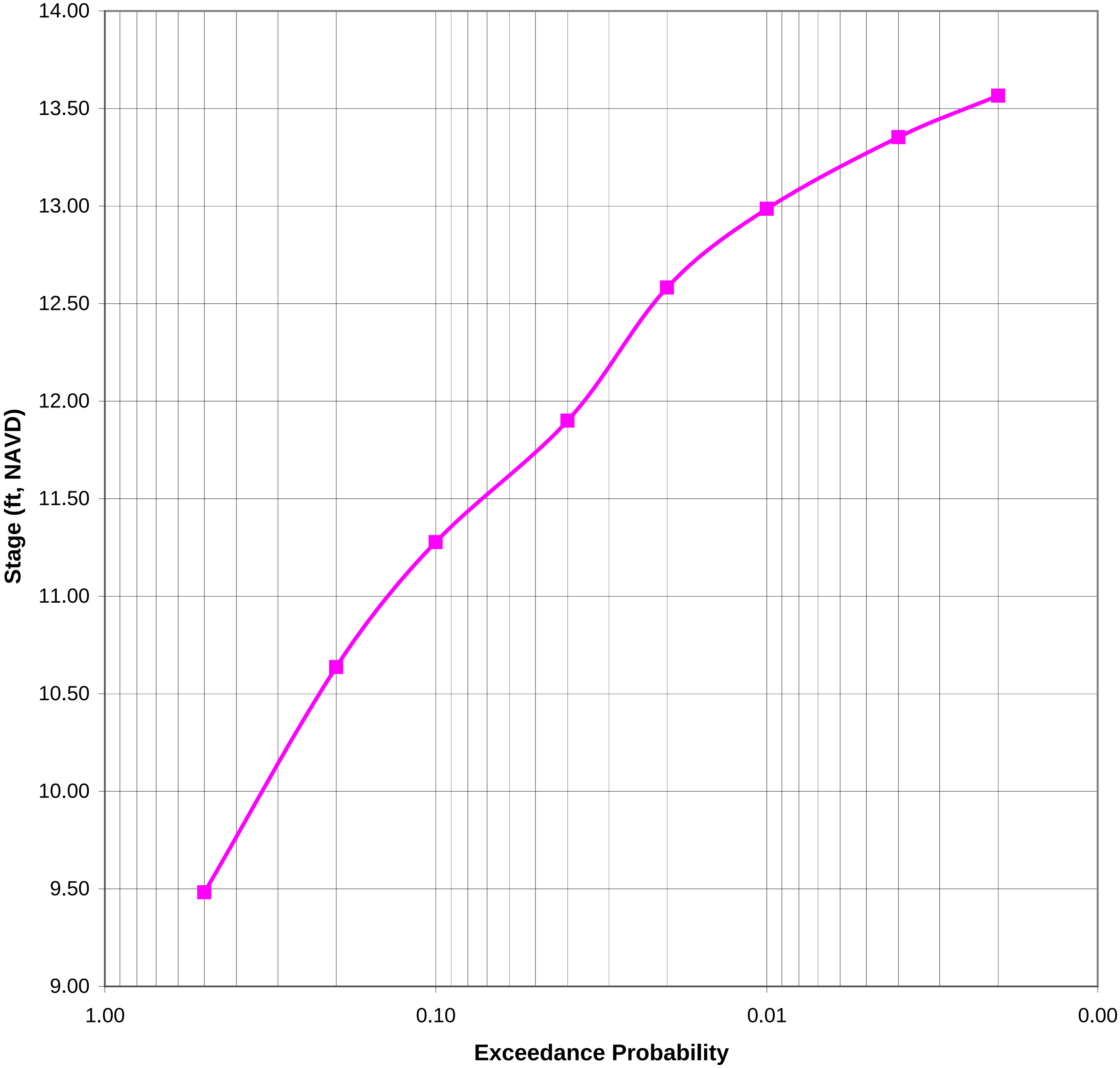
Legend:

Adobe & Barron Creeks, River Station 12565

South San Francisco Bay Shoreline Study
Alameda and Santa Clara Counties

Adobe & Barron Creeks,
River Station 12565, YEAR 0
Coincident Frequency Analysis Results

US Army Corps of Engineers
San Francisco District



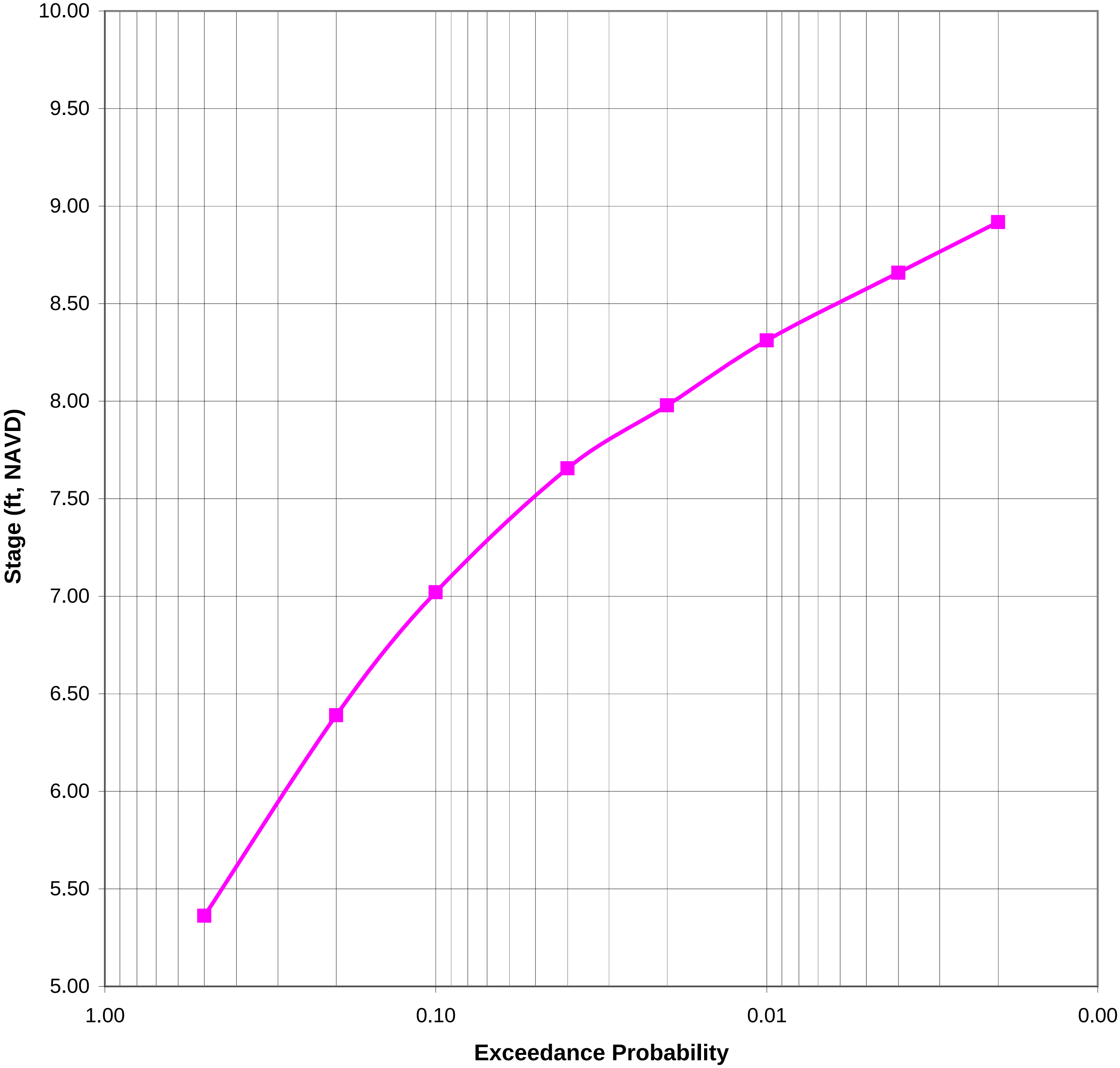
Legend:

Coyote Creek, River Station 7365

South San Francisco Bay Shoreline Study
Alameda and Santa Clara Counties

Coyote Creek,
River Station 7365, YEAR 0
Coincident Frequency Analysis Results

US Army Corps of Engineers
San Francisco District



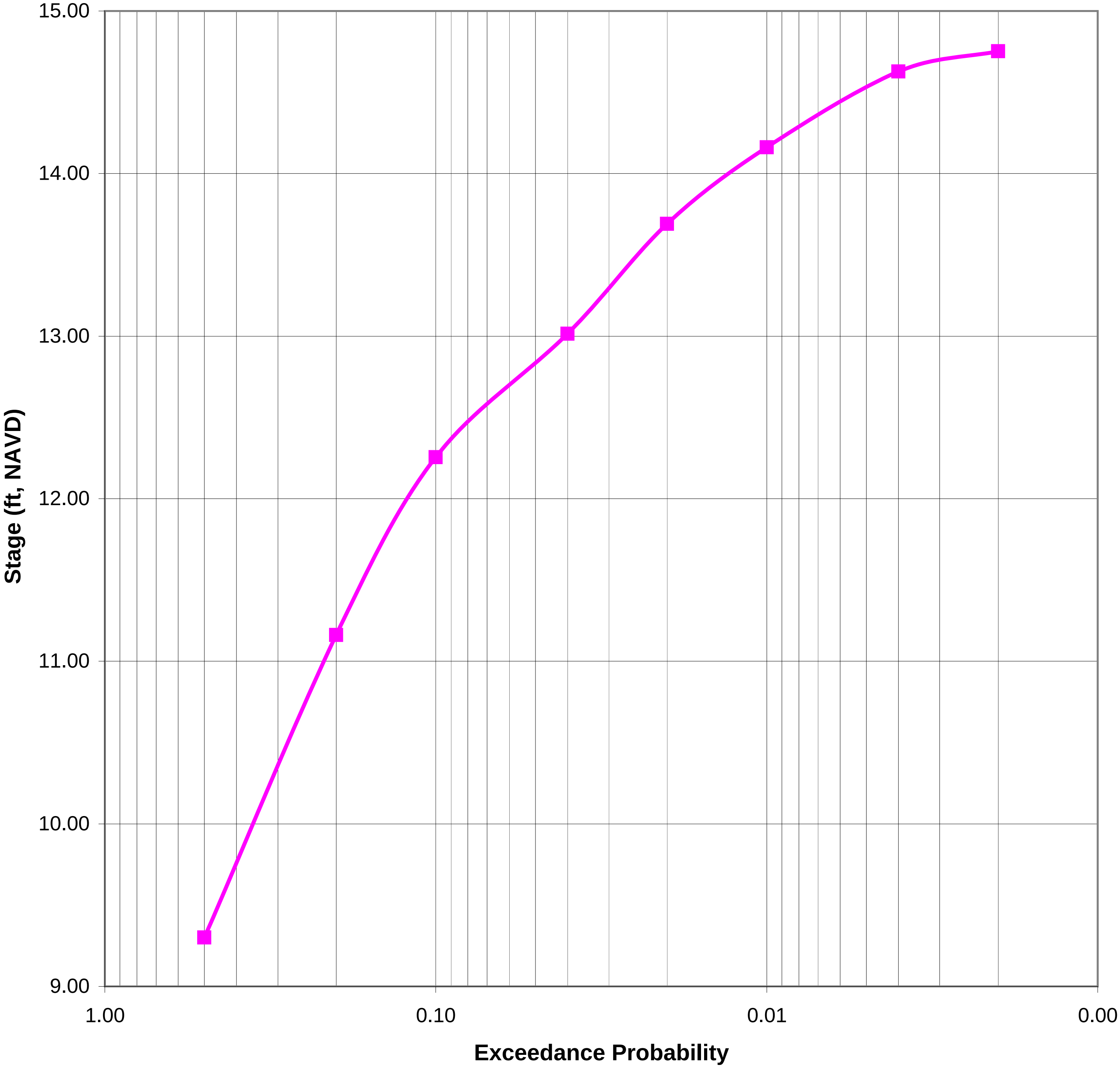
Legend:

Fremont Flood Control Channel, River Station 2404

South San Francisco Bay Shoreline Study
Alameda and Santa Clara Counties

Fremont Flood Control Channel, River Station 2404, YEAR 0 Coincident Frequency Analysis Results

US Army Corps of Engineers
San Francisco District



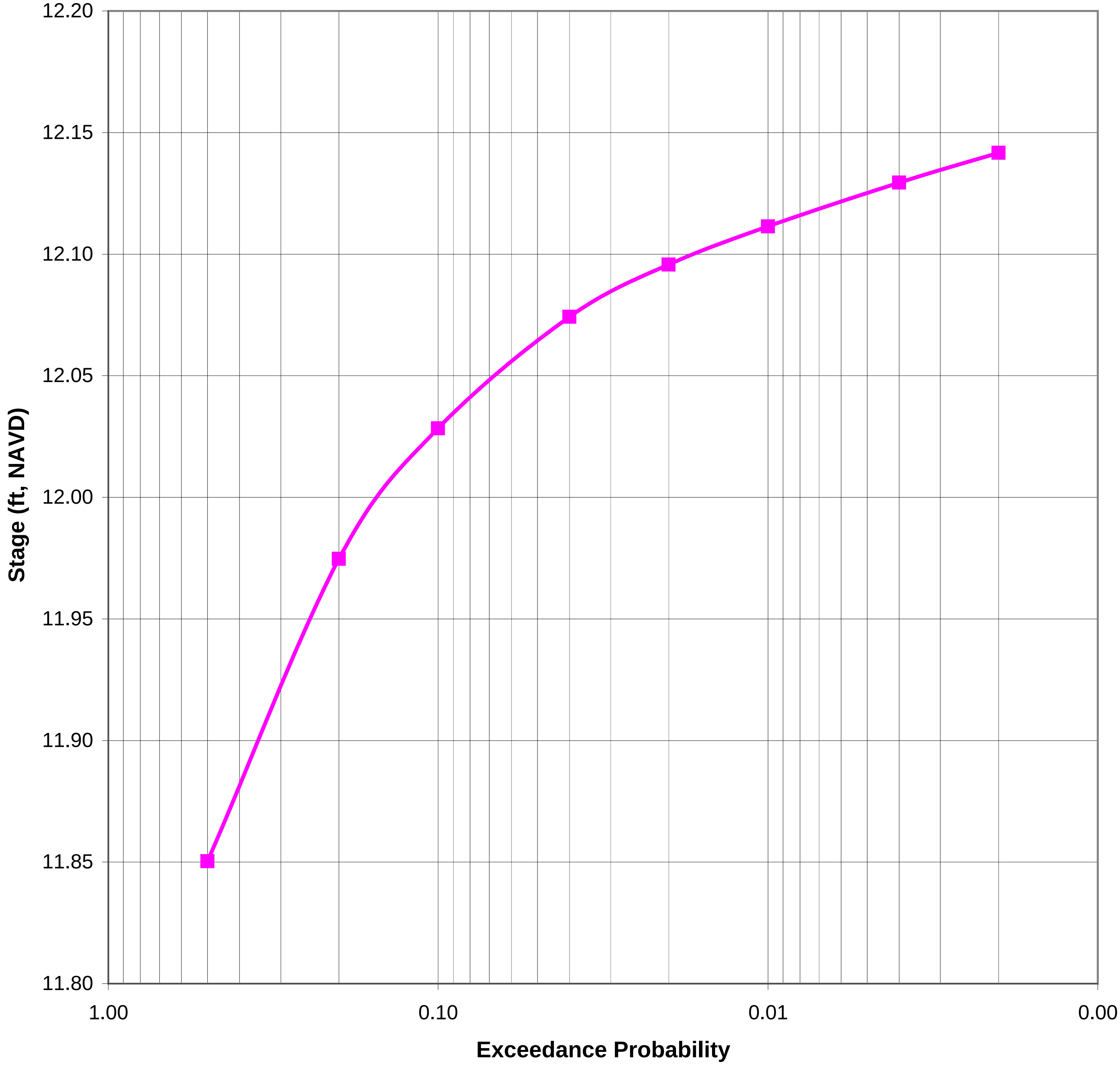
Legend:

Guadalupe River, Station 24481.23

South San Francisco Bay Shoreline Study
Alameda and Santa Clara Counties

Guadalupe River,
River Station 24481.23, YEAR 0
Coincident Frequency Analysis Results

US Army Corps of Engineers
San Francisco District



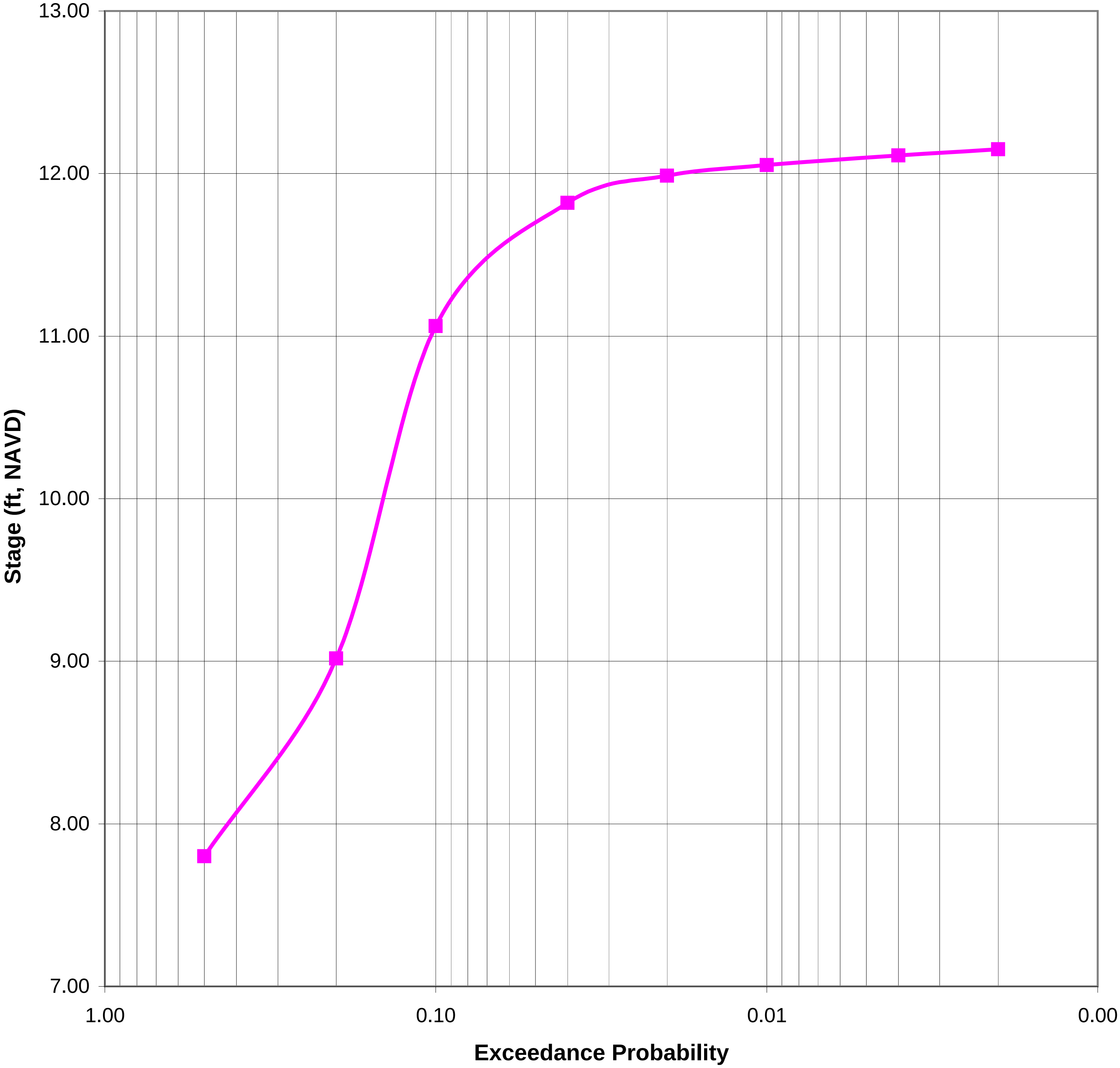
Legend:

Guadalupe Slough, River Station 27774

South San Francisco Bay Shoreline Study
Alameda and Santa Clara Counties

Guadalupe Slough,
River Station 27774, YEAR 0
Coincident Frequency Analysis Results

US Army Corps of Engineers
San Francisco District



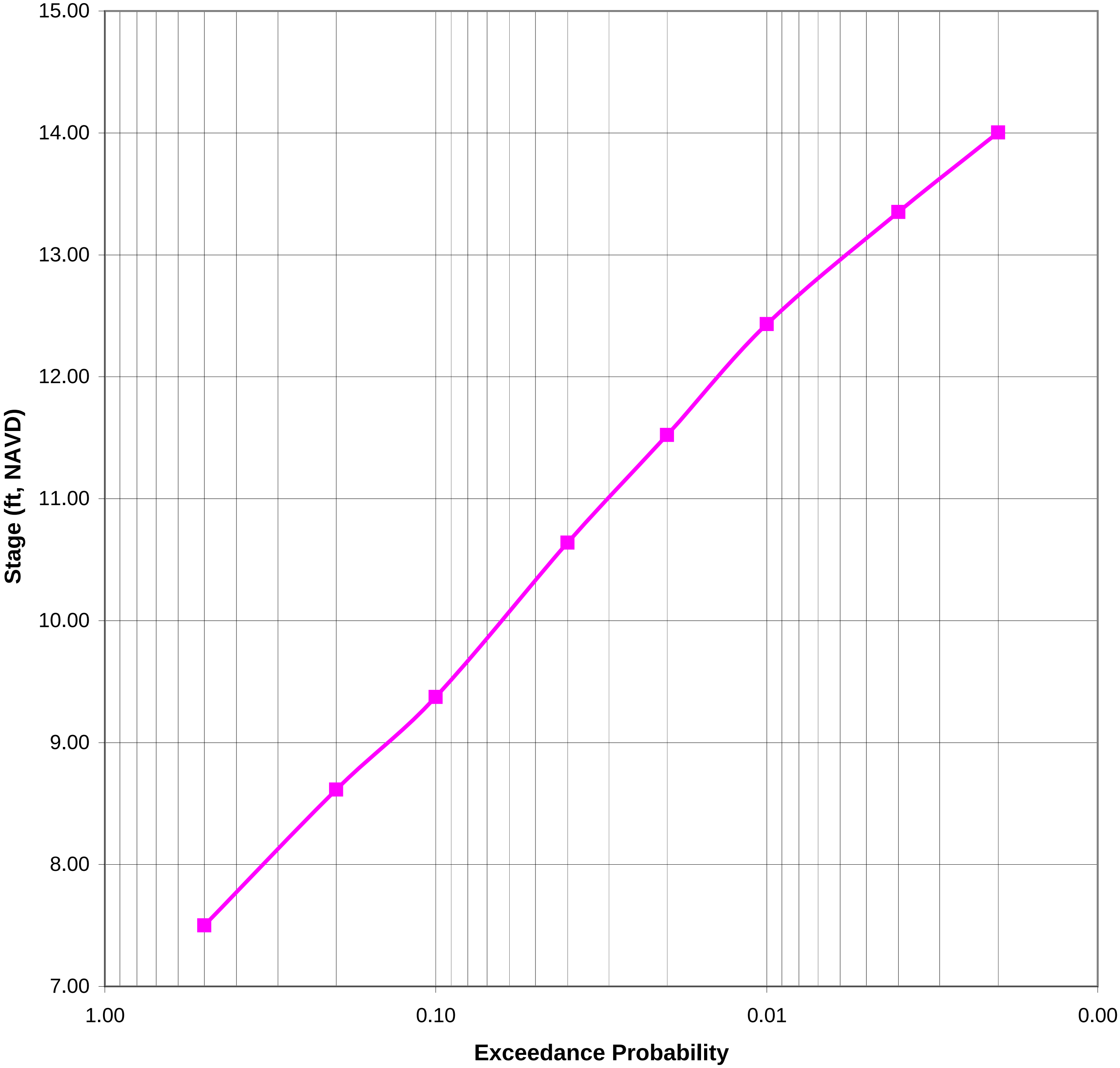
Legend:

Laguna Creek, River Station 19248

South San Francisco Bay Shoreline Study
Alameda and Santa Clara Counties

Laguna Creek,
River Station 19248, YEAR 0
Coincident Frequency Analysis Results

US Army Corps of Engineers
San Francisco District



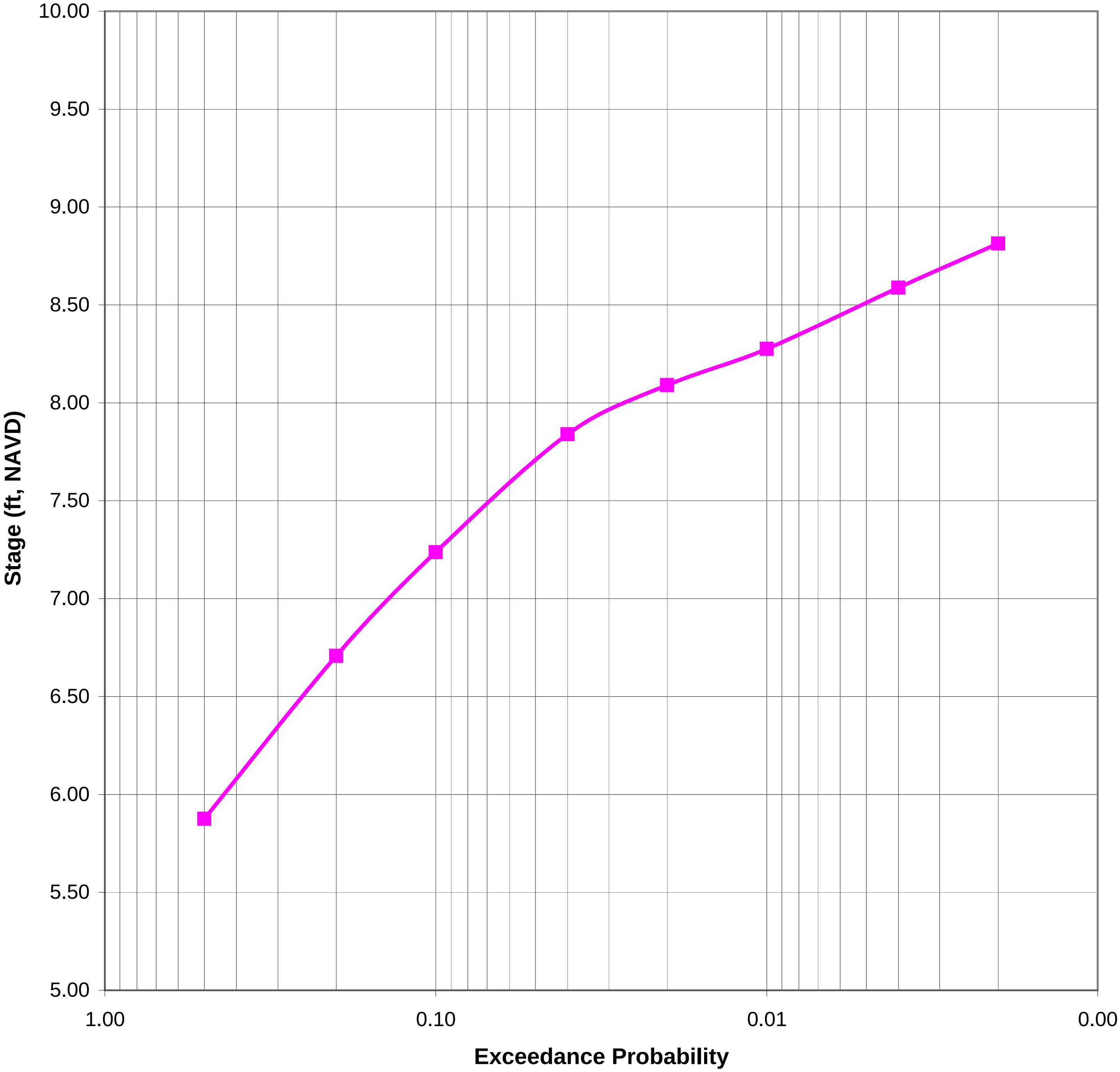
Legend:

Lower Penitencia Creek, River Station 376

South San Francisco Bay Shoreline Study
Alameda and Santa Clara Counties

Lower Penitencia Creek,
River Station 376, YEAR 0
Coincident Frequency Analysis Results

US Army Corps of Engineers
San Francisco District



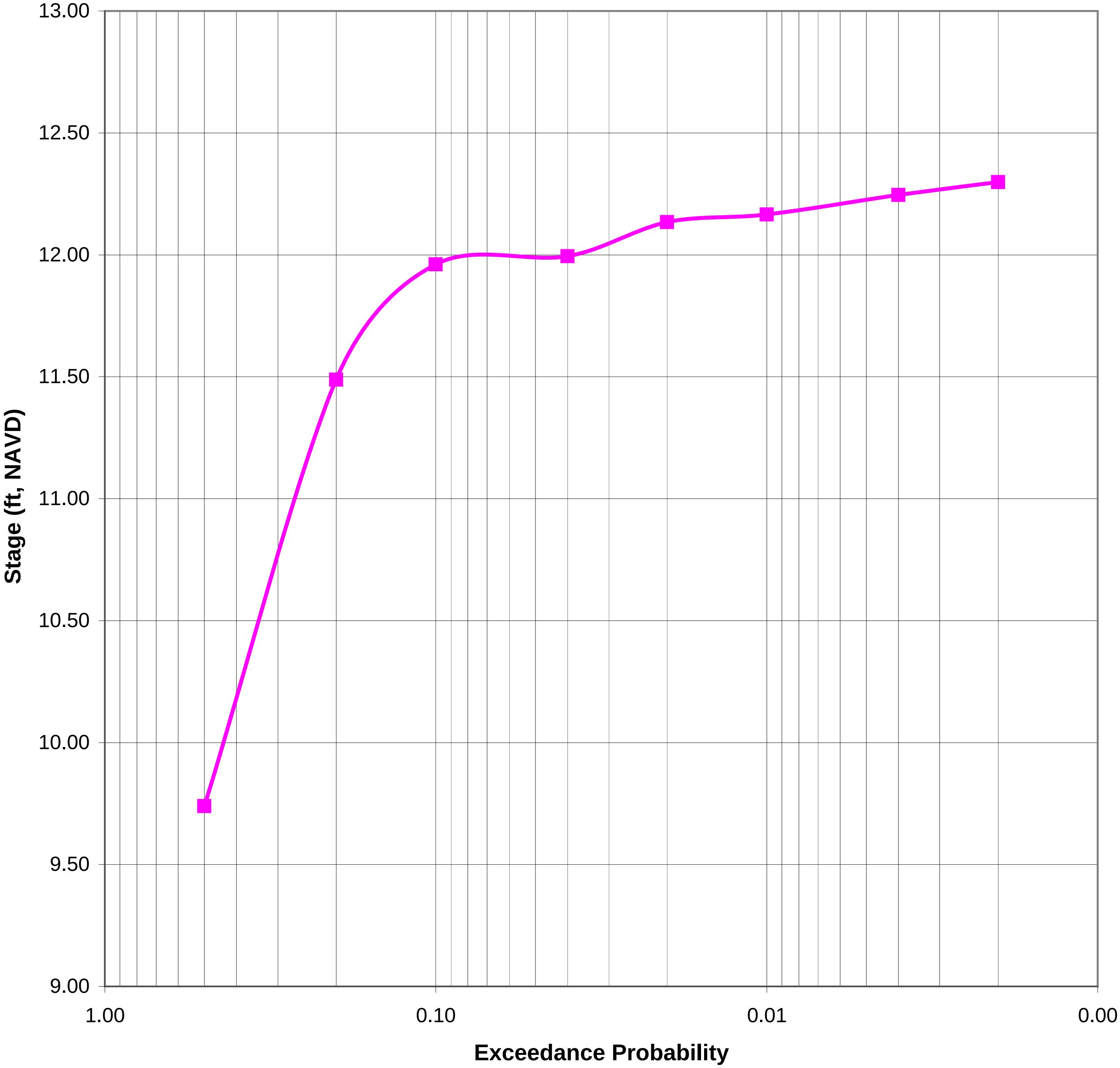
Legend:

Matadero Creek, River Station 9983

South San Francisco Bay Shoreline Study
Alameda and Santa Clara Counties

Matadero Creek,
River Station 9983, YEAR 0
Coincident Frequency Analysis Results

US Army Corps of Engineers
San Francisco District



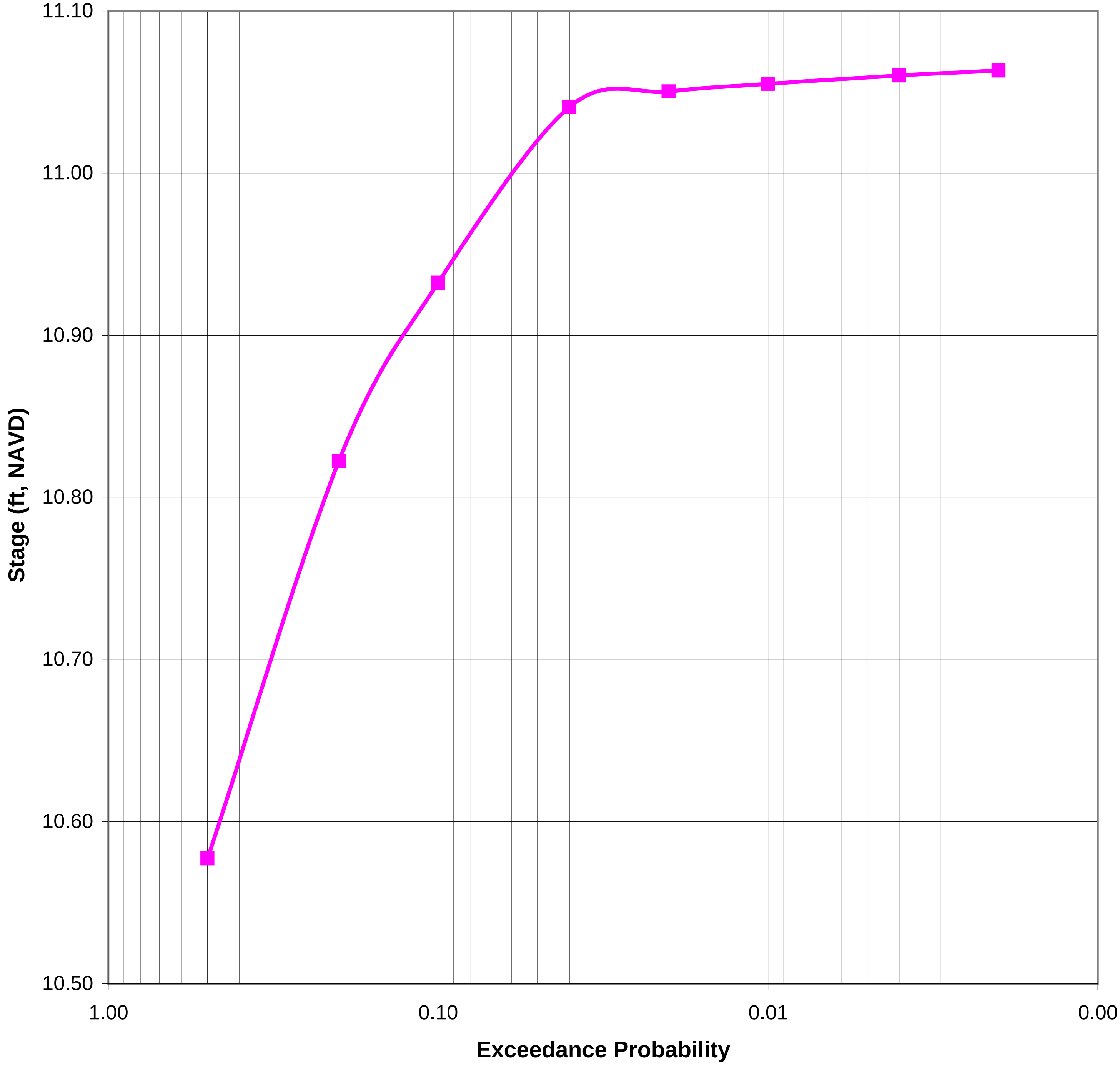
Legend:

Permanente Creek, River Station 8793.68

South San Francisco Bay Shoreline Study
Alameda and Santa Clara Counties

Permanente Creek,
River Station 8793.68, YEAR 0
Coincident Frequency Analysis Results

US Army Corps of Engineers
San Francisco District



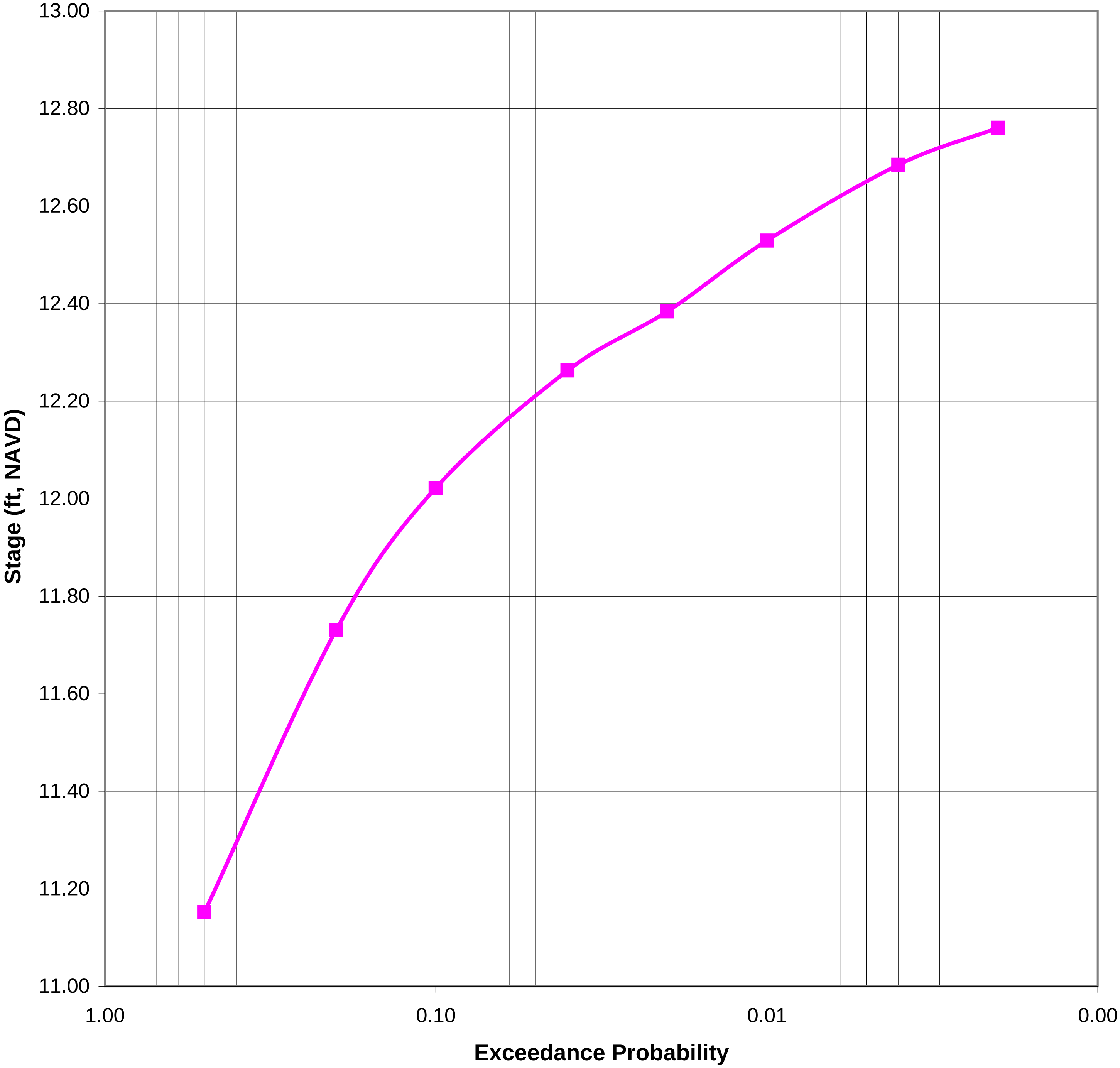
Legend:

Stevens Creek, River Station 4896

South San Francisco Bay Shoreline Study
Alameda and Santa Clara Counties

Stevens Creek,
River Station 4896, YEAR 0
Coincident Frequency Analysis Results

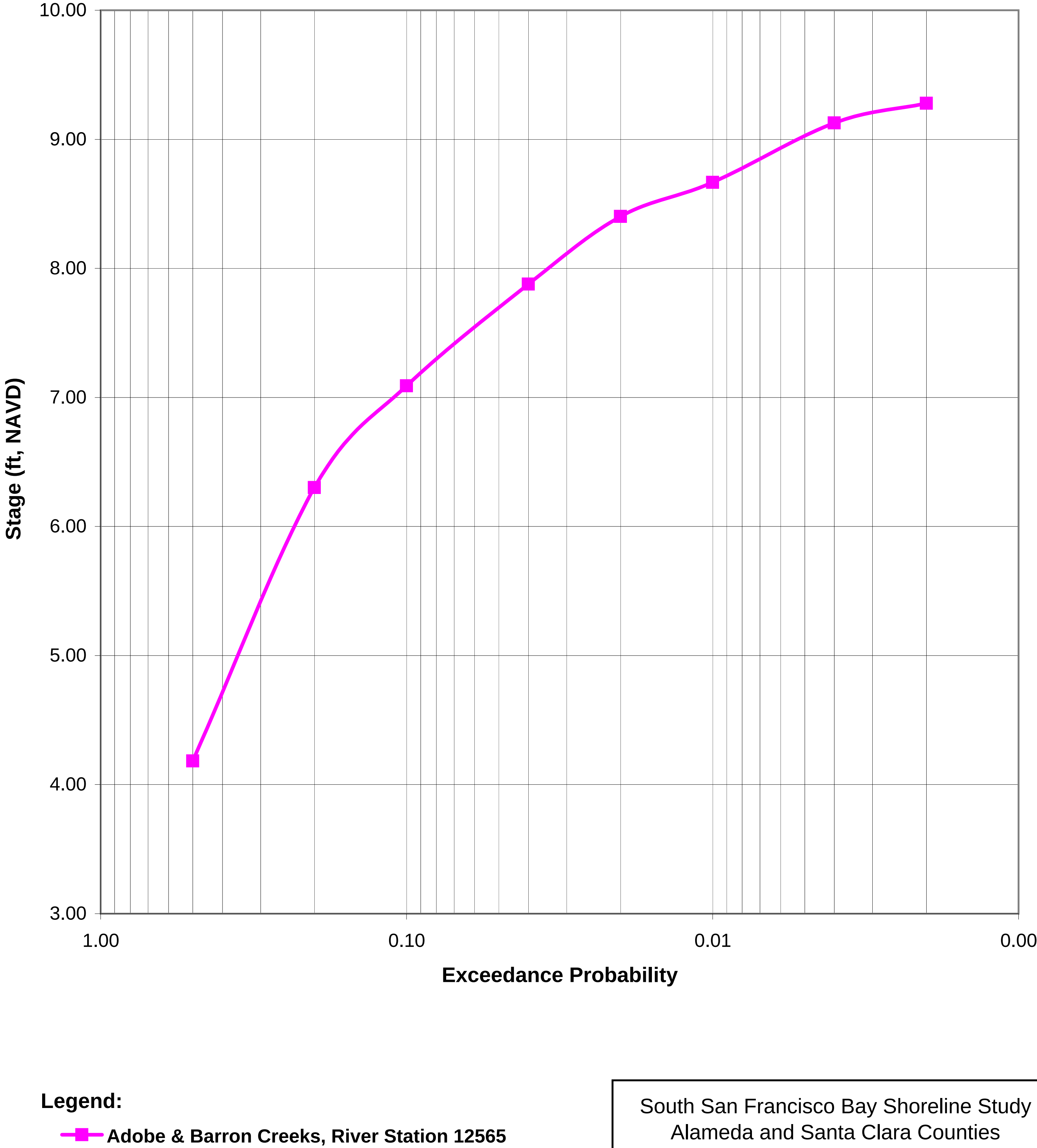
US Army Corps of Engineers
San Francisco District



Legend:

Sunnyvale West Channel, River Station 10652

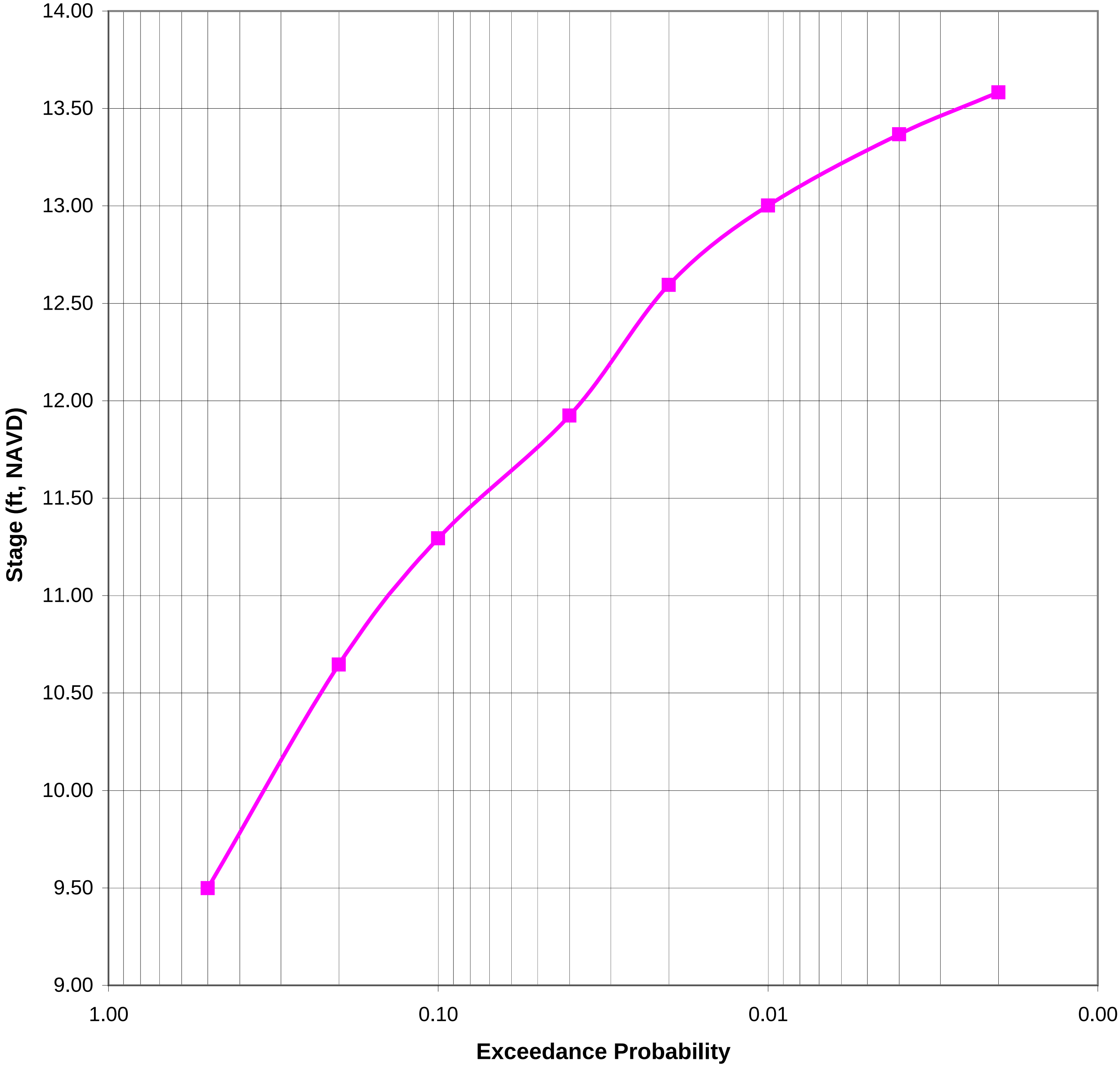
South San Francisco Bay Shoreline Study Alameda and Santa Clara Counties
Sunnyvale West Channel, River Station 10652, YEAR 0 Coincident Frequency Analysis Results
US Army Corps of Engineers San Francisco District



South San Francisco Bay Shoreline Study
Alameda and Santa Clara Counties

Adobe & Barron Creeks,
River Station 12565, YEAR 50
Coincident Frequency Analysis Results

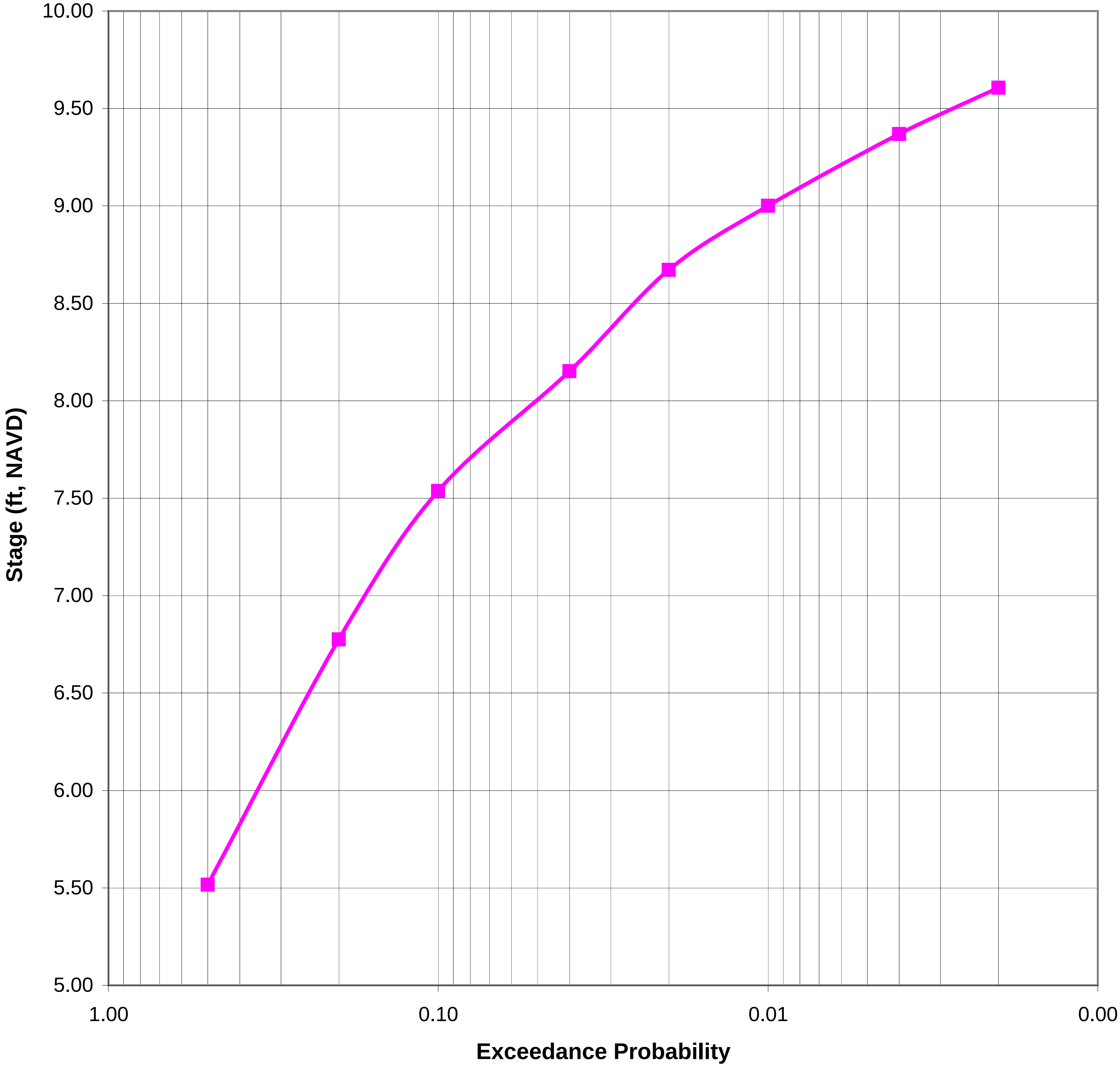
US Army Corps of Engineers
San Francisco District



Legend:

Coyote Creek, River Station 7365

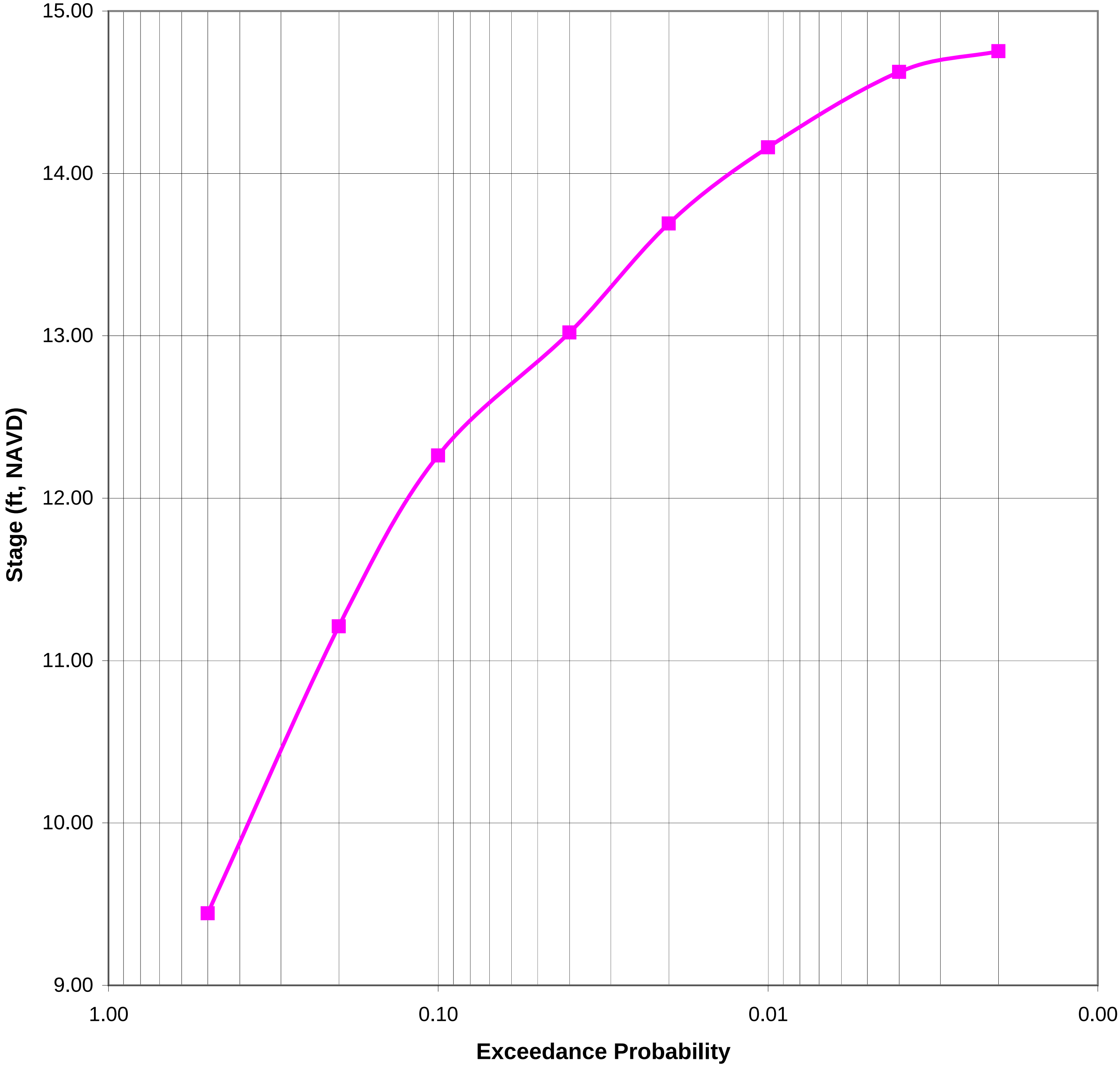
South San Francisco Bay Shoreline Study Alameda and Santa Clara Counties
Coyote Creek, River Station 7365, YEAR 50 Coincident Frequency Analysis Results
US Army Corps of Engineers San Francisco District



Legend:

Fremont Flood Control Channel, River Station 2404

South San Francisco Bay Shoreline Study Alameda and Santa Clara Counties
Fremont Flood Control Channel, River Station 2404, YEAR 50 Coincident Frequency Analysis Results
US Army Corps of Engineers San Francisco District



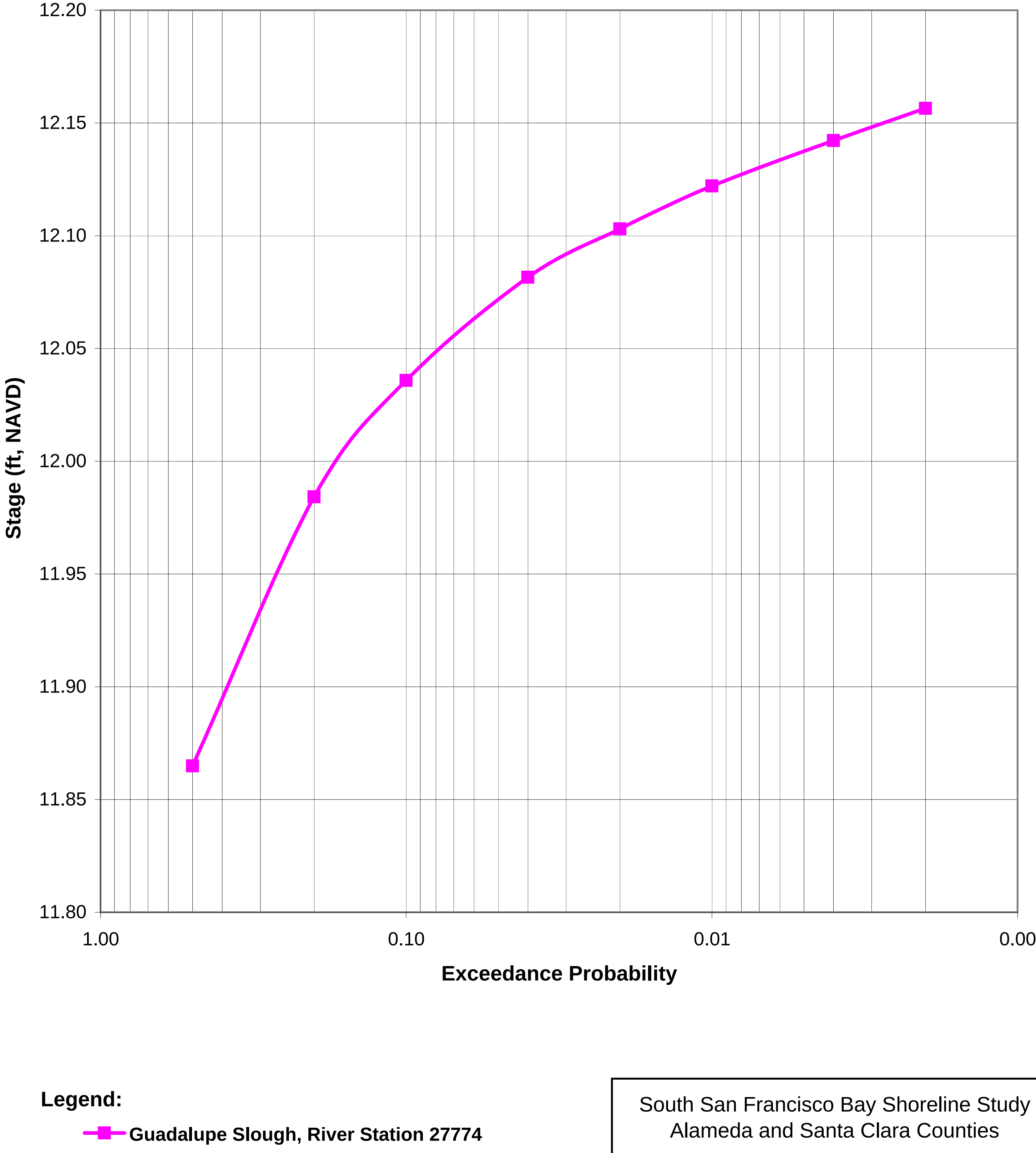
Legend:

Guadalupe River, Station 24481.23

South San Francisco Bay Shoreline Study
Alameda and Santa Clara Counties

Guadalupe River,
River Station 24481.23, YEAR 50
Coincident Frequency Analysis Results

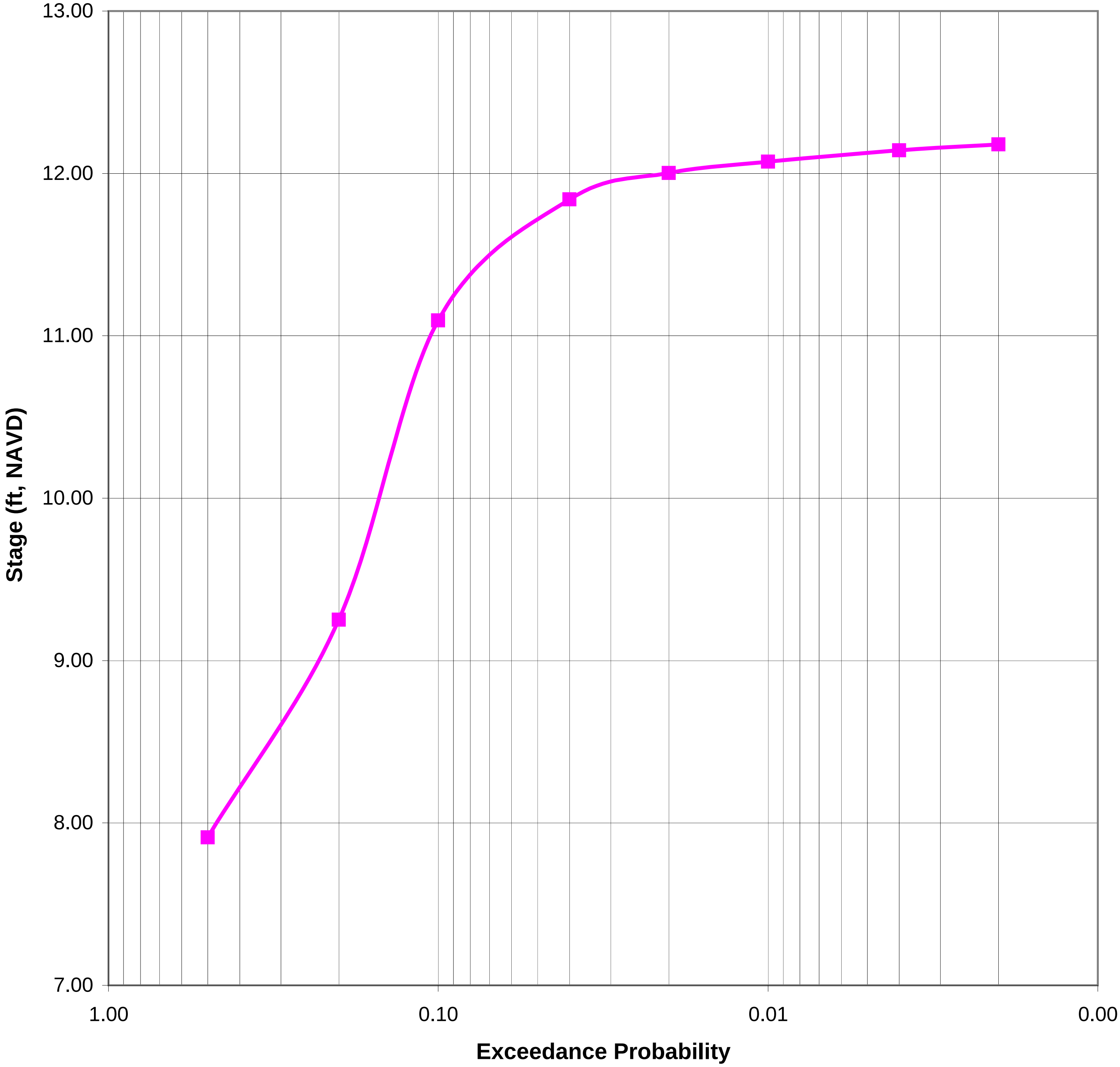
US Army Corps of Engineers
San Francisco District



South San Francisco Bay Shoreline Study
Alameda and Santa Clara Counties

Guadalupe Slough,
River Station 27774, YEAR 50
Coincident Frequency Analysis Results

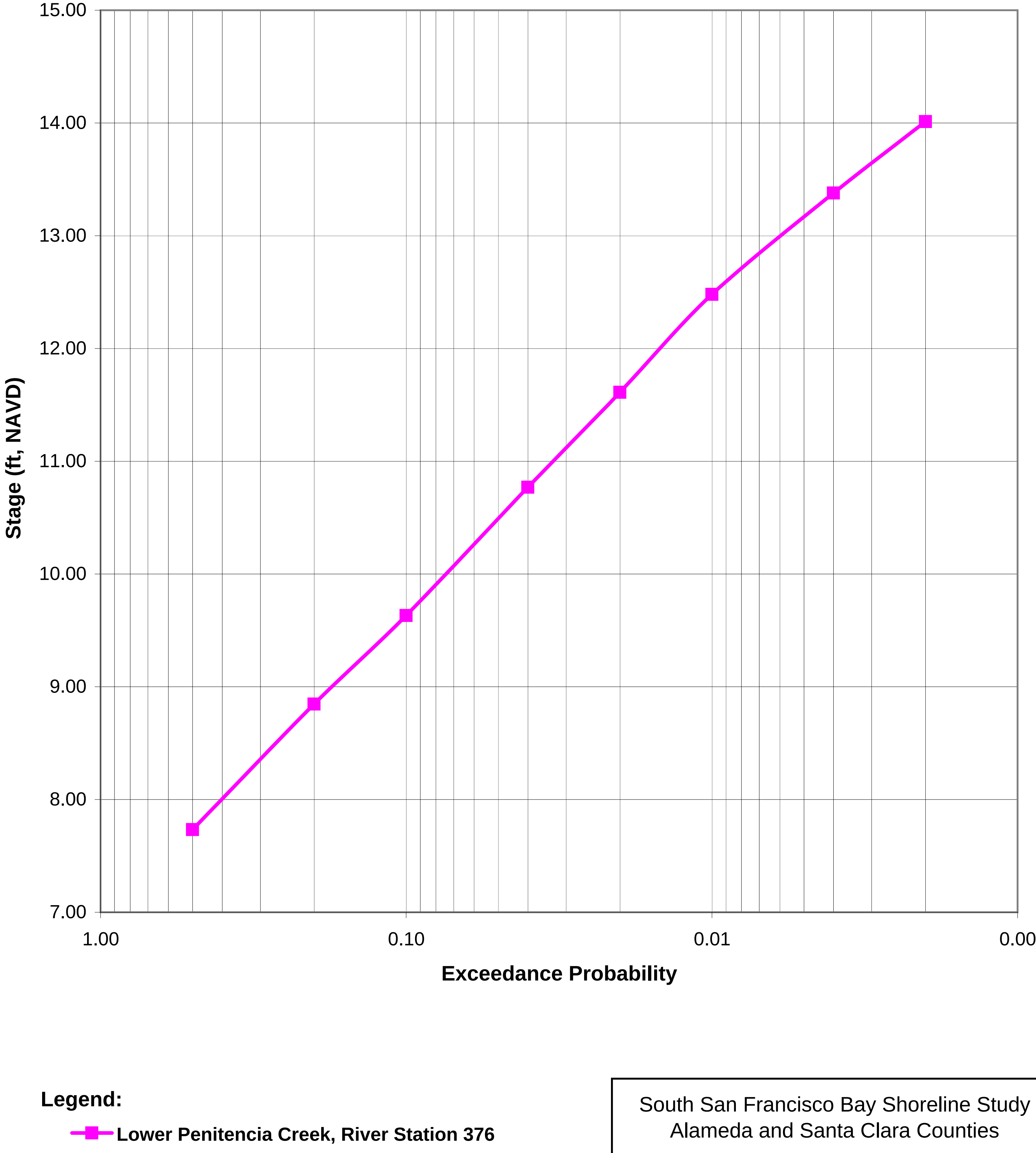
US Army Corps of Engineers
San Francisco District



Legend:

Laguna Creek, River Station 19248

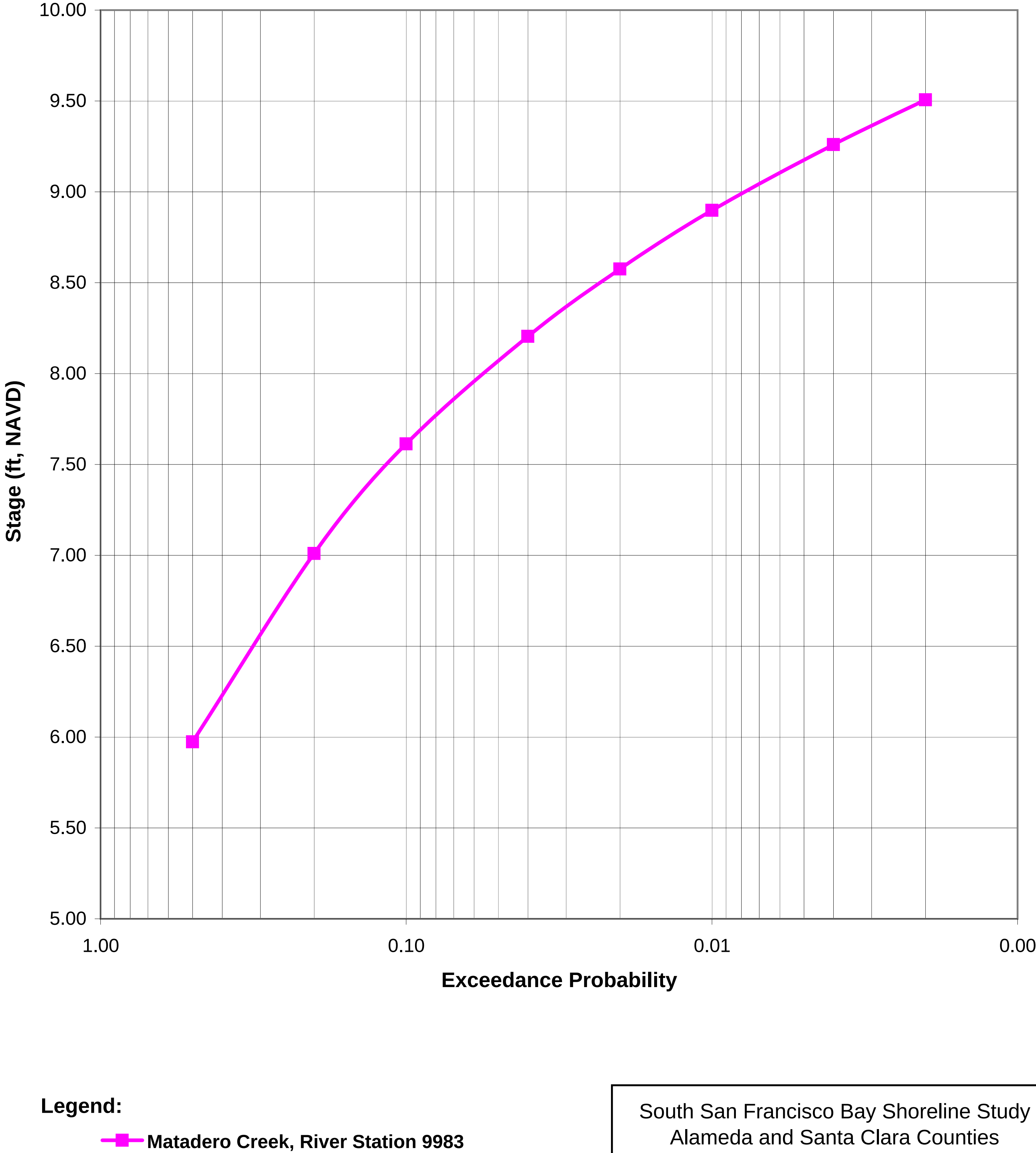
South San Francisco Bay Shoreline Study Alameda and Santa Clara Counties
Laguna Creek, River Station 19248, YEAR 50 Coincident Frequency Analysis Results
US Army Corps of Engineers San Francisco District



South San Francisco Bay Shoreline Study
Alameda and Santa Clara Counties

Lower Penitencia Creek,
River Station 376, YEAR 50
Coincident Frequency Analysis Results

US Army Corps of Engineers
San Francisco District

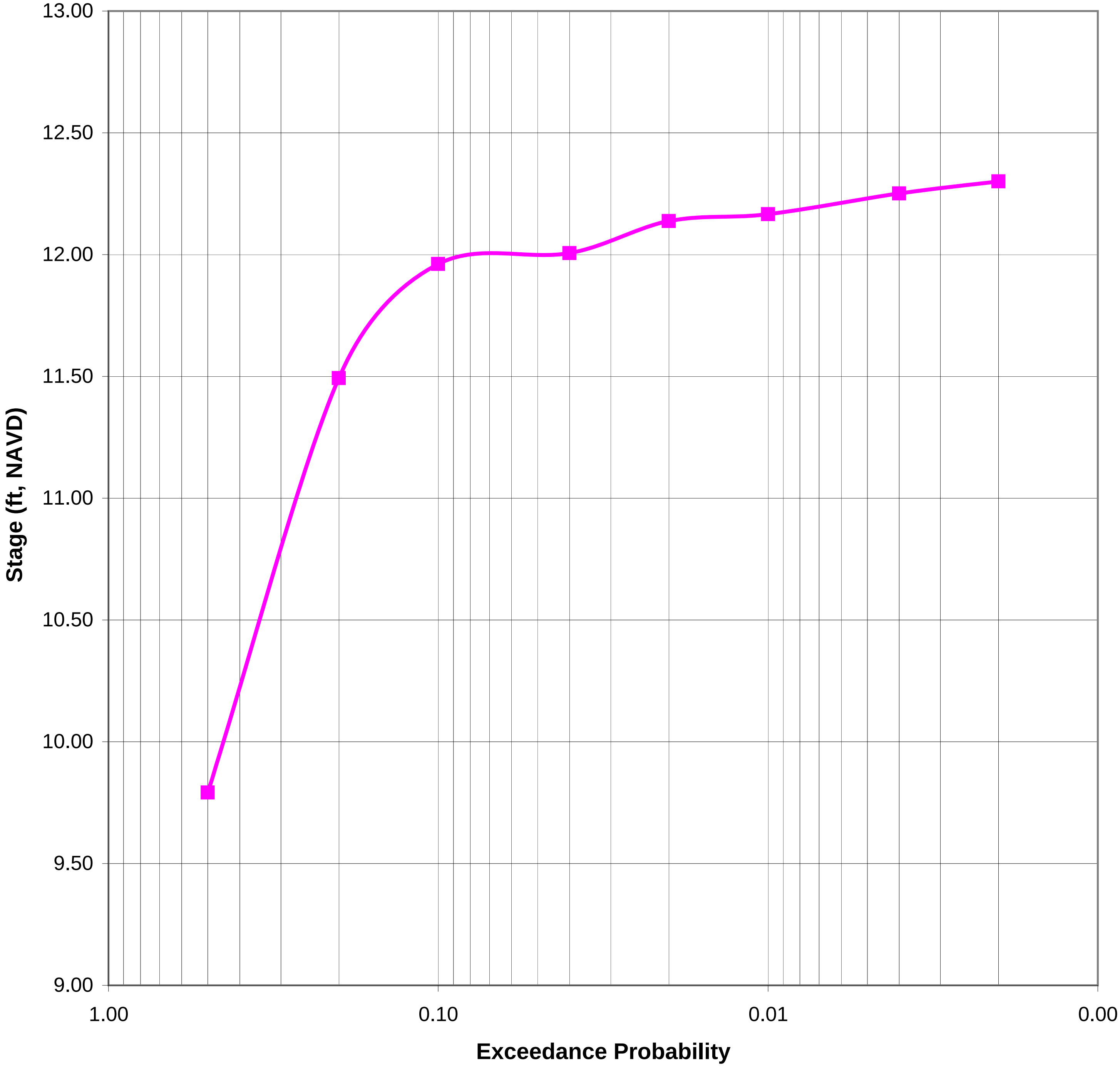


South San Francisco Bay Shoreline Study
Alameda and Santa Clara Counties

Matadero Creek,
River Station 9983, YEAR 50
Coincident Frequency Analysis Results

US Army Corps of Engineers
San Francisco District

PLATE 44h



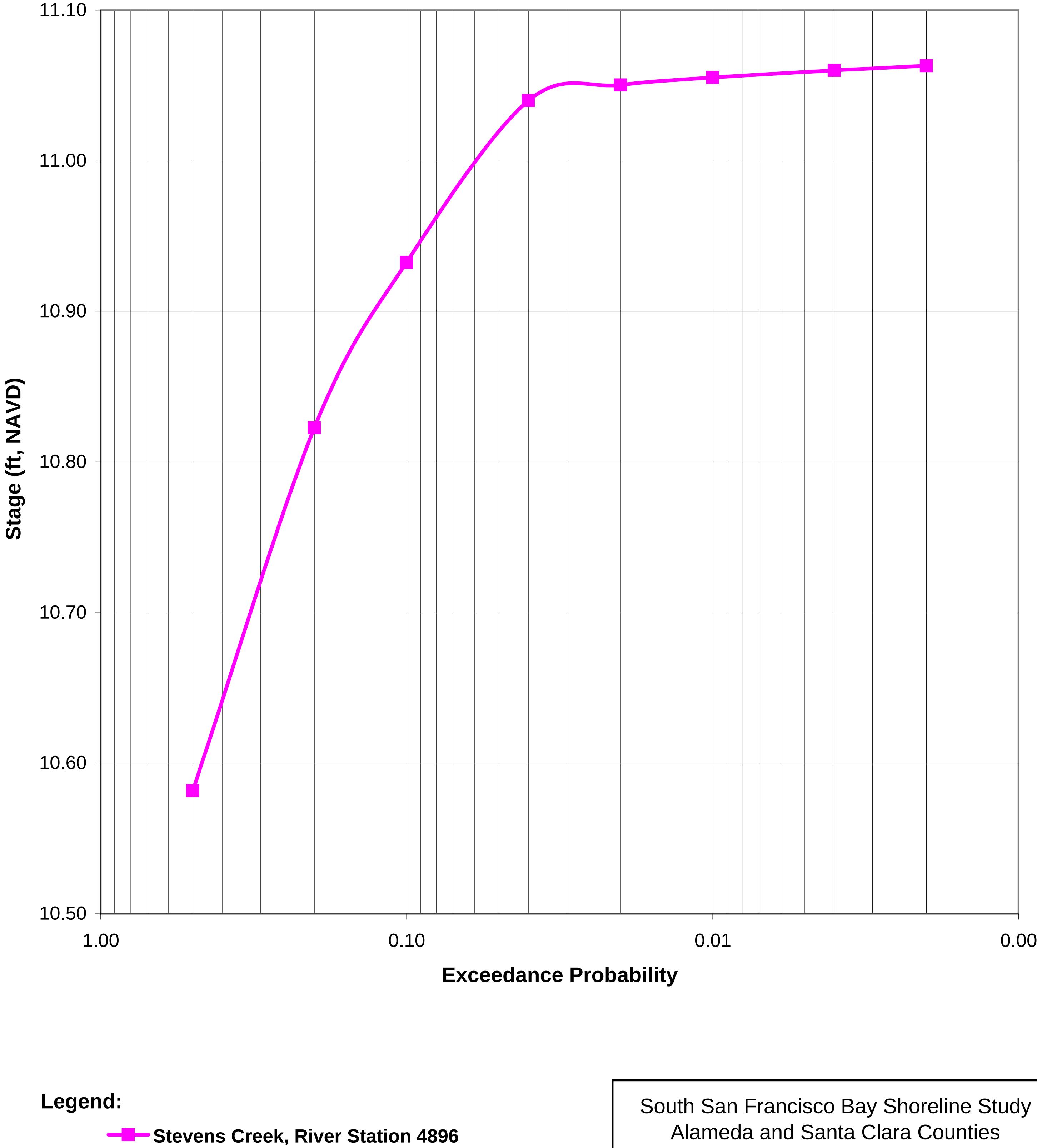
Legend:

Permanente Creek, River Station 8793.68

South San Francisco Bay Shoreline Study
Alameda and Santa Clara Counties

Permanente Creek,
River Station 8793.68, YEAR 50
Coincident Frequency Analysis Results

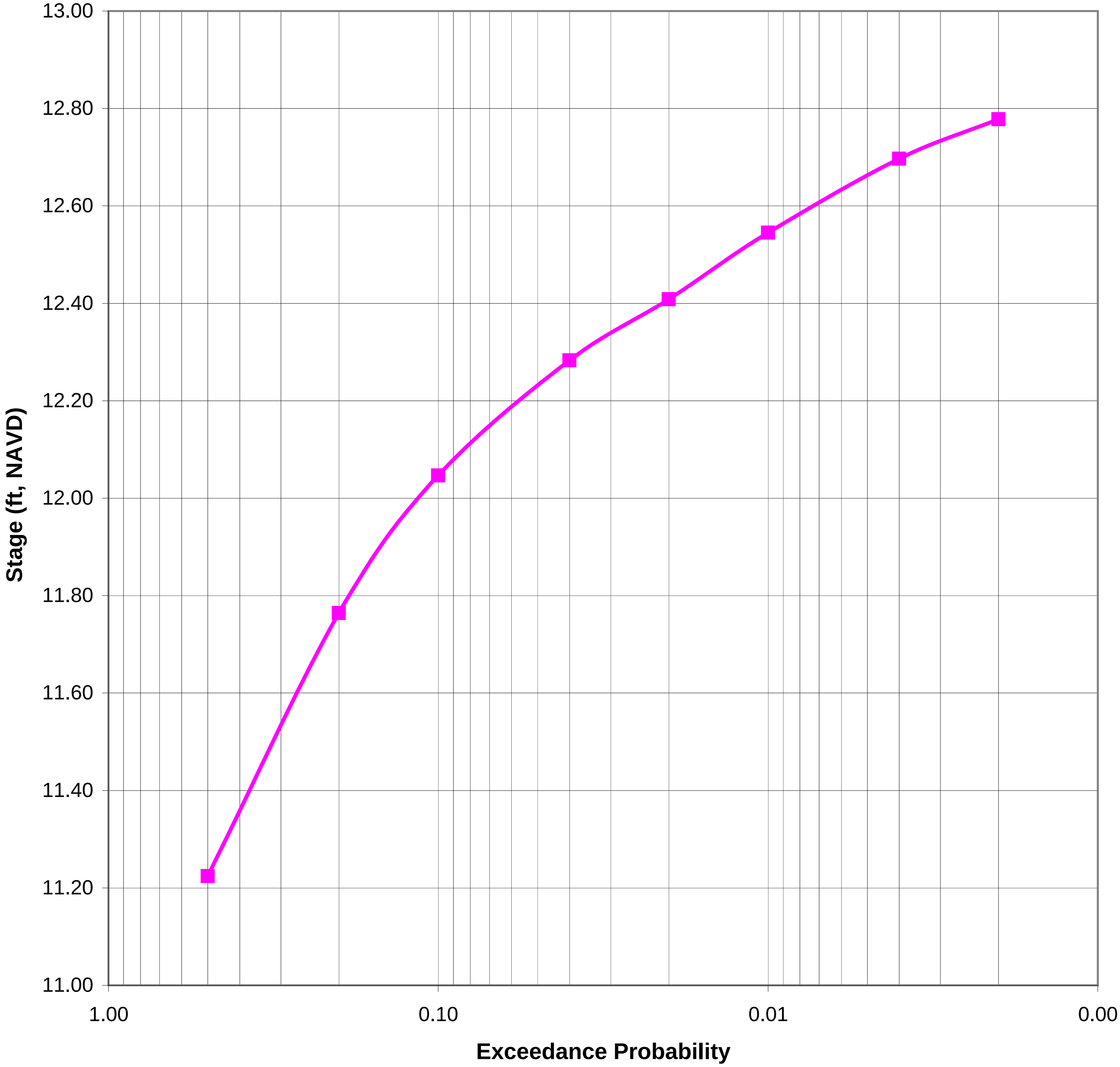
US Army Corps of Engineers
San Francisco District



South San Francisco Bay Shoreline Study
Alameda and Santa Clara Counties

Stevens Creek,
River Station 4896, YEAR 50
Coincident Frequency Analysis Results

US Army Corps of Engineers
San Francisco District



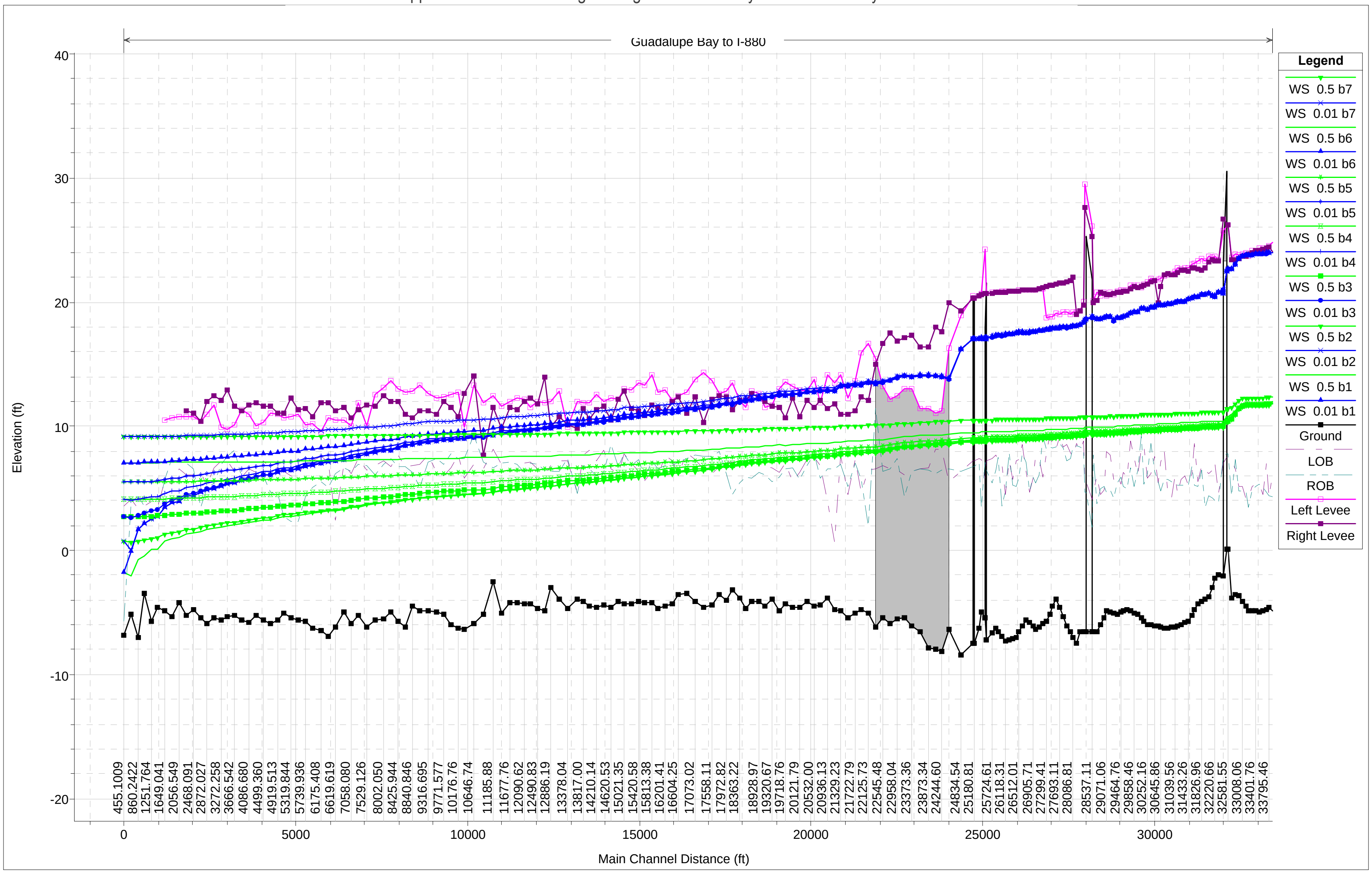
Legend:

Sunnyvale West Channel, River Station 10652

South San Francisco Bay Shoreline Study
Alameda and Santa Clara Counties

Sunnyvale West Channel,
River Station 10652, YEAR 50
Coincident Frequency Analysis Results

US Army Corps of Engineers
San Francisco District

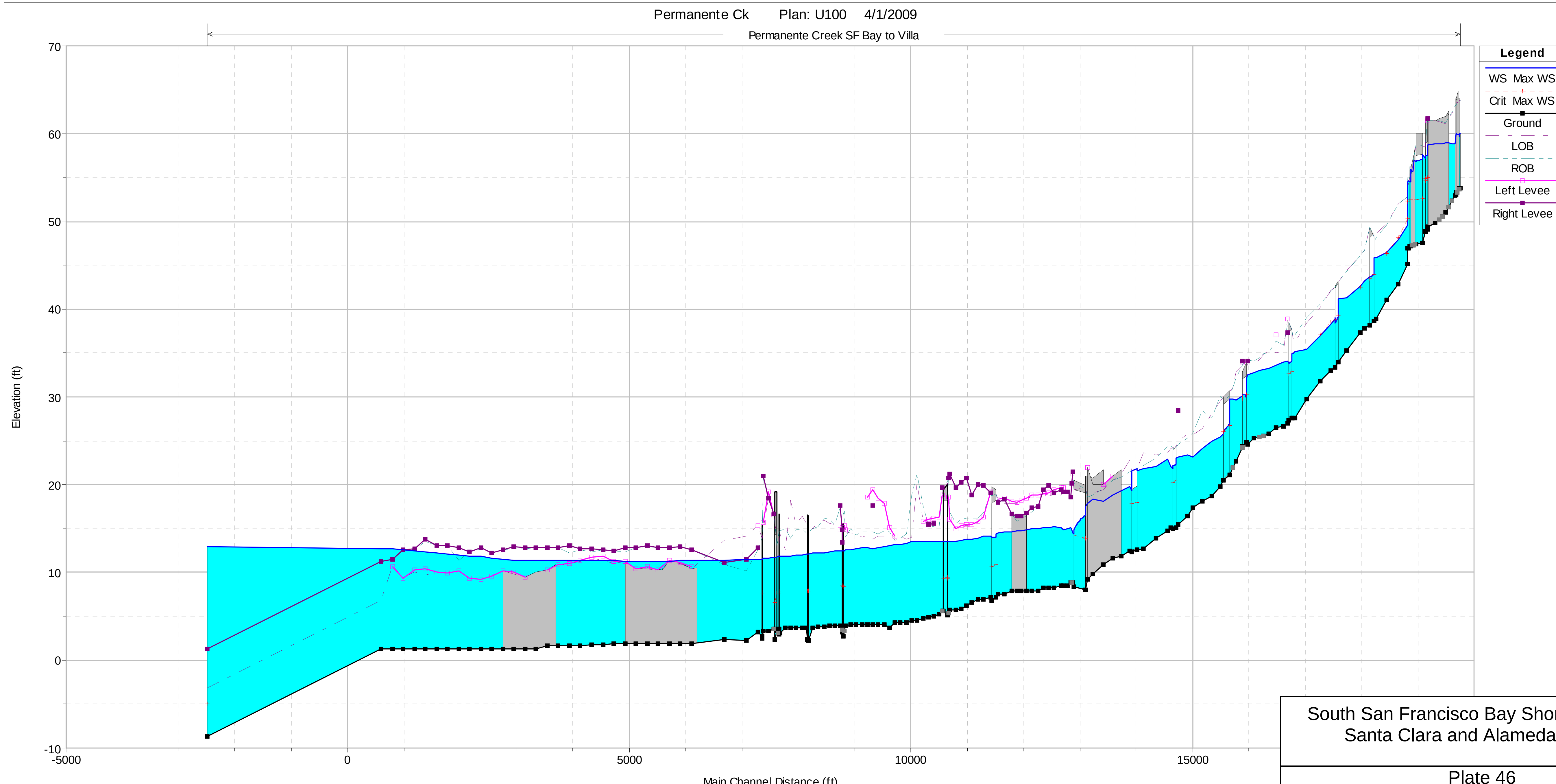


South San Francisco Bay Shoreline Study,
Santa Clara and Alameda County

Coincident Frequency Analysis
Example of Tidal and Flow Influence
on Downstream Boudary Condition

US Army Corps of Engineers
San Francisco District

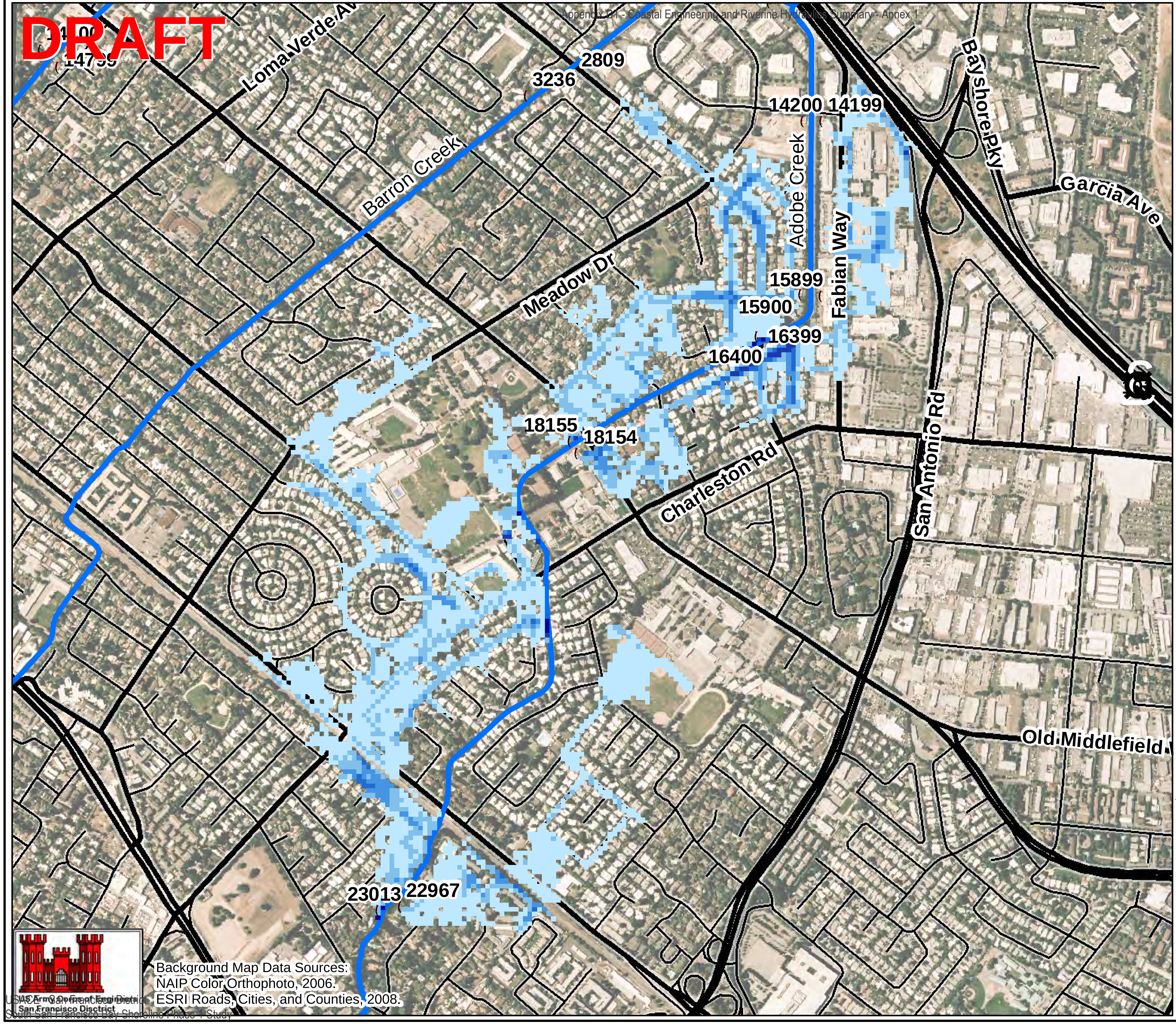
PLATE 45



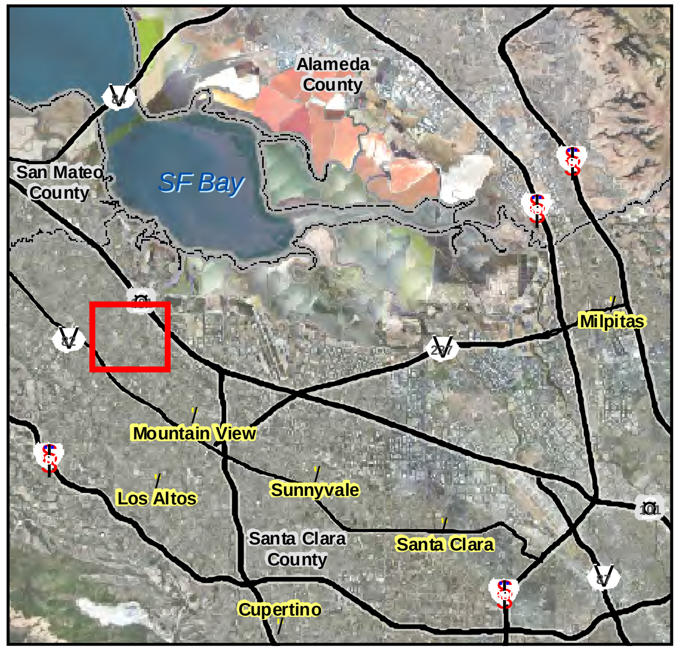
South San Francisco Bay Shoreline Study, Santa Clara and Alameda County

Plate 46
Example of Longitudinal Profile and Additional Cross Section

US Army Corps of Engineers
San Francisco District



Regional Reference Map



Scale - 1:10,000

Scale - 1 in = 833.3 feet

0 625 1,250 2,500 Feet

State Plane CA Zone III	horz. datum: NAD 83	horz. units: feet
	vert. datum: NAVD 88	18 Jun 2010

Legend

- Breakout Locations (Stations Labeled)
- Stream Centerline
- Roads

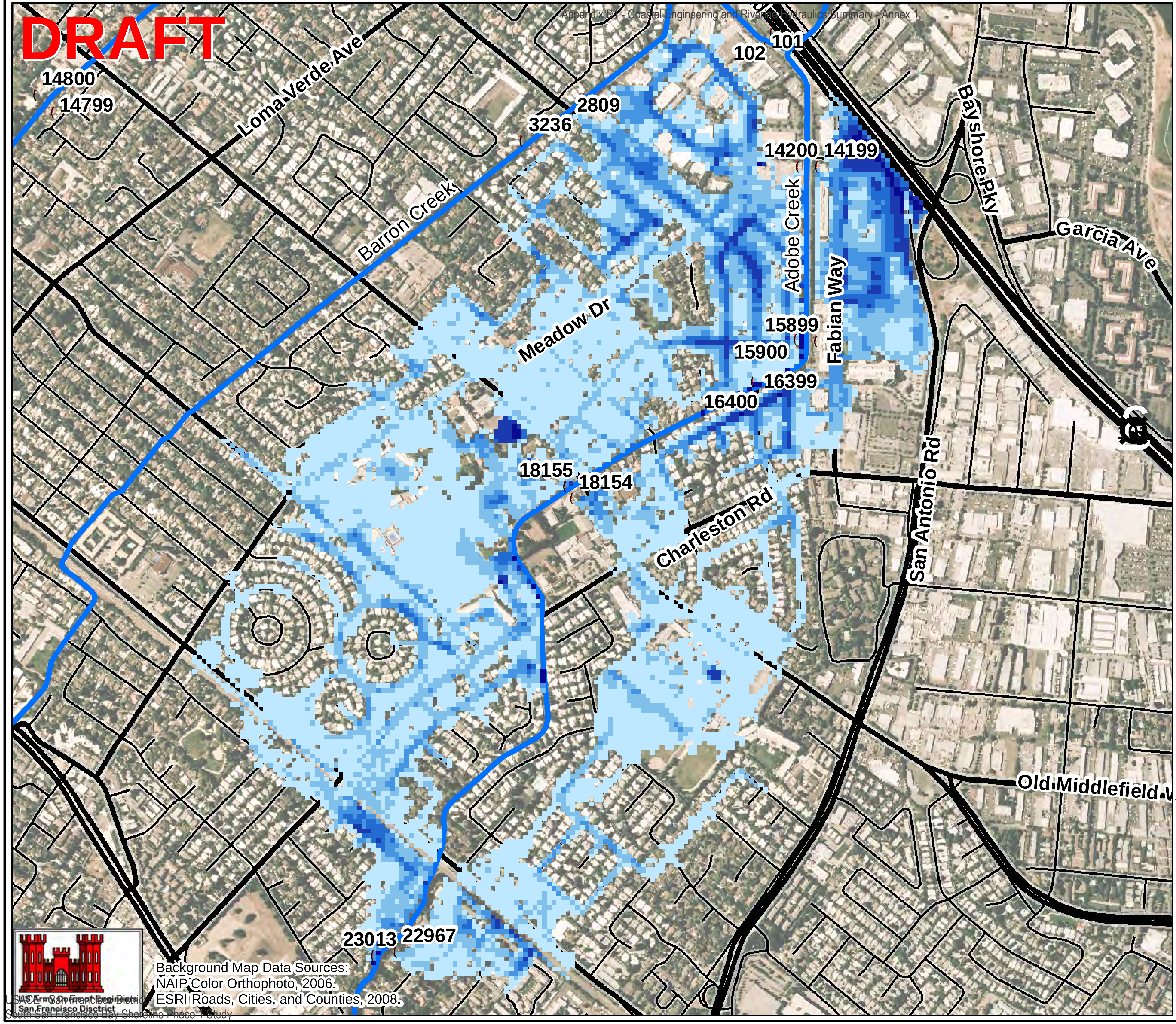
Max Water Depth (ft)

- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 1.5
- 1.5 - 2.0
- 2.0 - 3.0
- 3.0 - 9.0

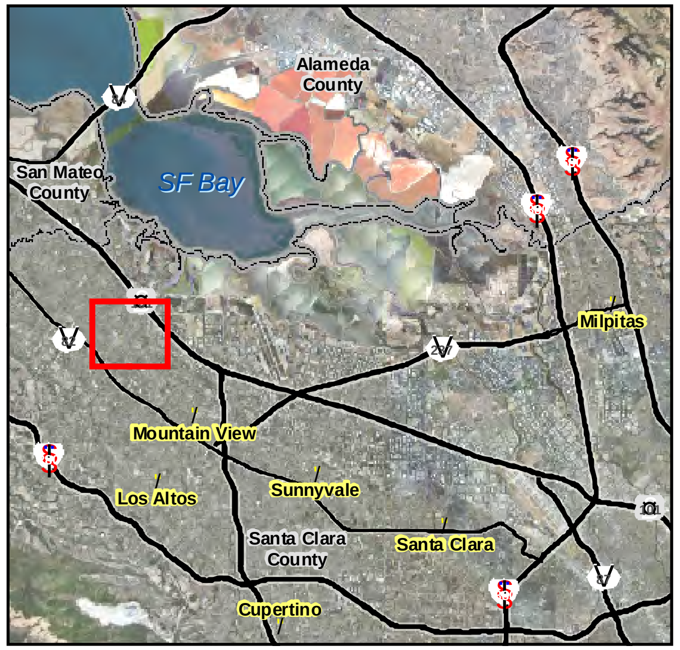
1% Exceedance Probability Fluvial Flood Inundation Map

Adobe Creek Plate 47

US Army Corps of Engineers
San Francisco District



Regional Reference Map



Scale - 1:10,000

Scale - 1 in = 833.3 feet

0 625 1,250 2,500 Feet

State Plane CA Zone III	horz. datum: NAD 83 vert. datum: NAVD 88	horz. units: feet 18 Jun 2010

Legend

(Breakout Locations (Stations Labeled)

Stream Centerline

Roads

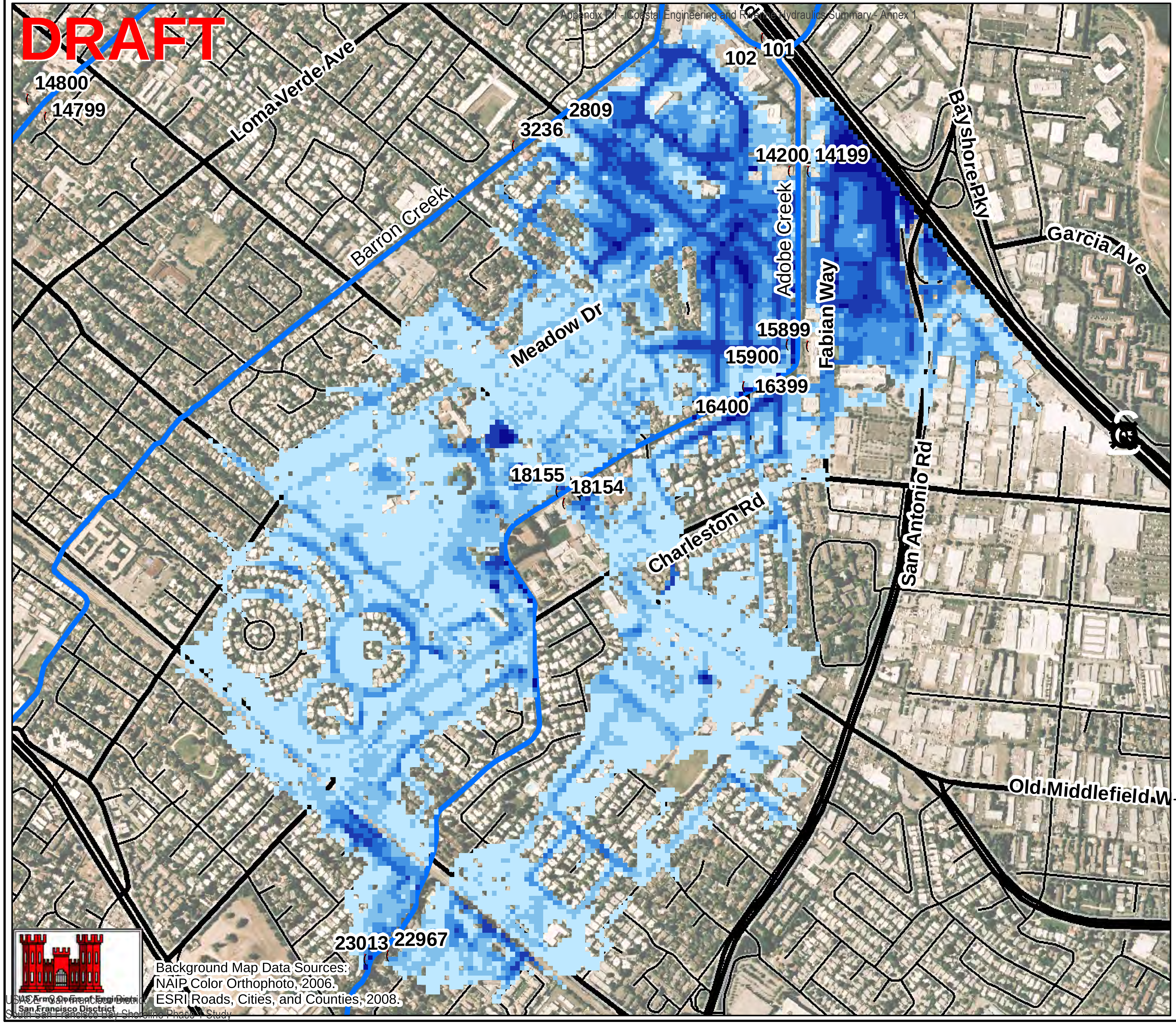
Max Water Depth (ft)

- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 1.5
- 1.5 - 2.0
- 2.0 - 3.0
- 3.0 - 9.0

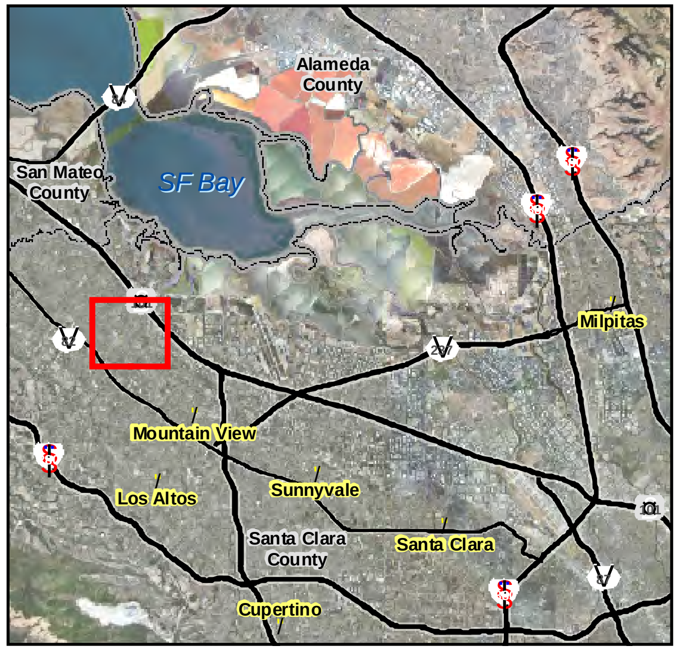
0.4% Exceedance Probability Fluvial Flood Inundation Map

Adobe Creek
Plate 48

US Army Corps of Engineers
San Francisco District



Regional Reference Map



Scale - 1:10,000

Scale - 1 in = 833.3 feet

0 625 1,250 2,500 Feet

State Plane CA Zone III	horz. datum: NAD 83 vert. datum: NAVD 88	horz. units: feet 18 Jun 2010

Legend

- Breakout Locations (Stations Labeled)
- Stream Centerline
- Roads

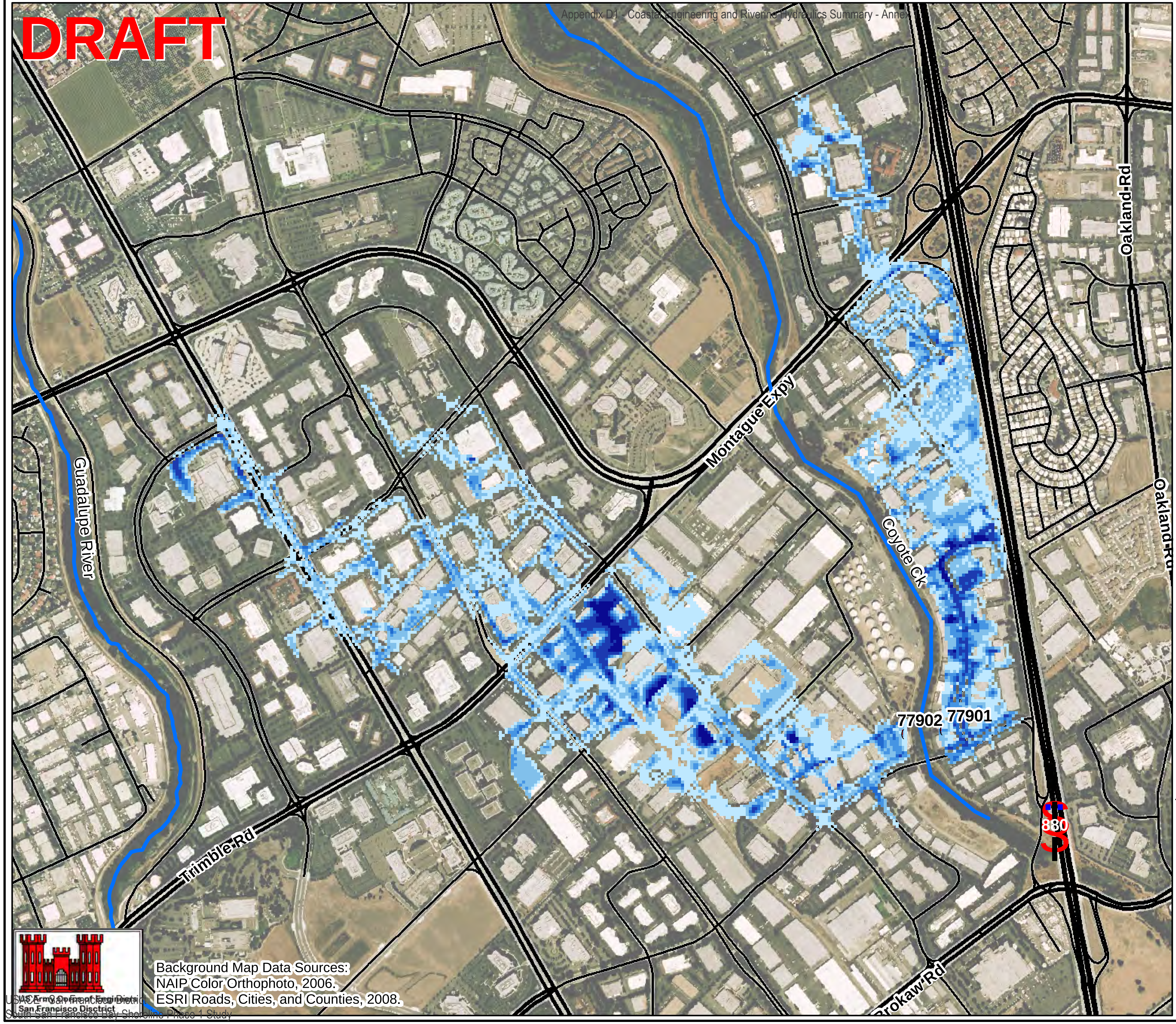
Max Water Depth (ft)

- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 1.5
- 1.5 - 2.0
- 2.0 - 3.0
- 3.0 - 9.0

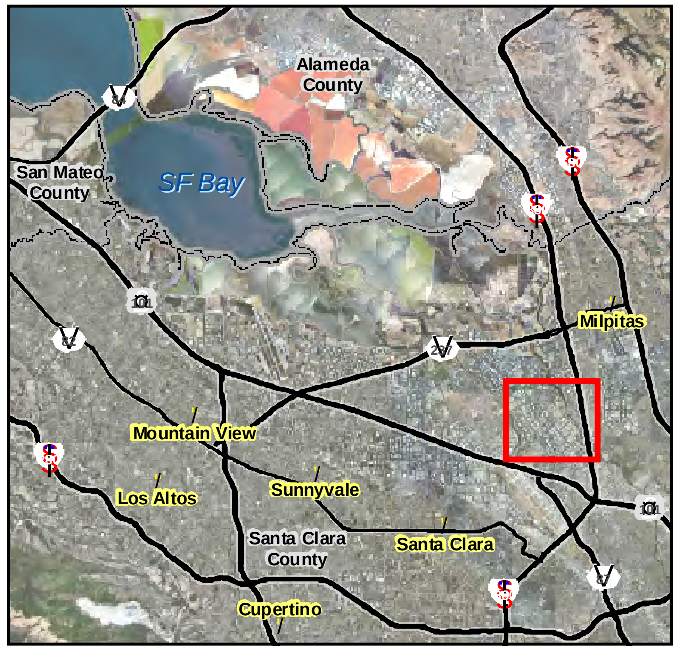
0.2% Exceedance Probability Fluvial Flood Inundation Map

Adobe Creek
Plate 49

US Army Corps of Engineers
San Francisco District



Regional Reference Map



Scale - 1:12,000

Scale - 1 in = 1,000 feet

0 750 1,500 3,000 Feet

State Plane CA Zone III	horz. datum: NAD 83 vert. datum: NAVD 88	horz. units: feet 18 Jun 2010

Legend

- Breakout Locations (Stations Labeled)
- Stream Centerline
- Roads

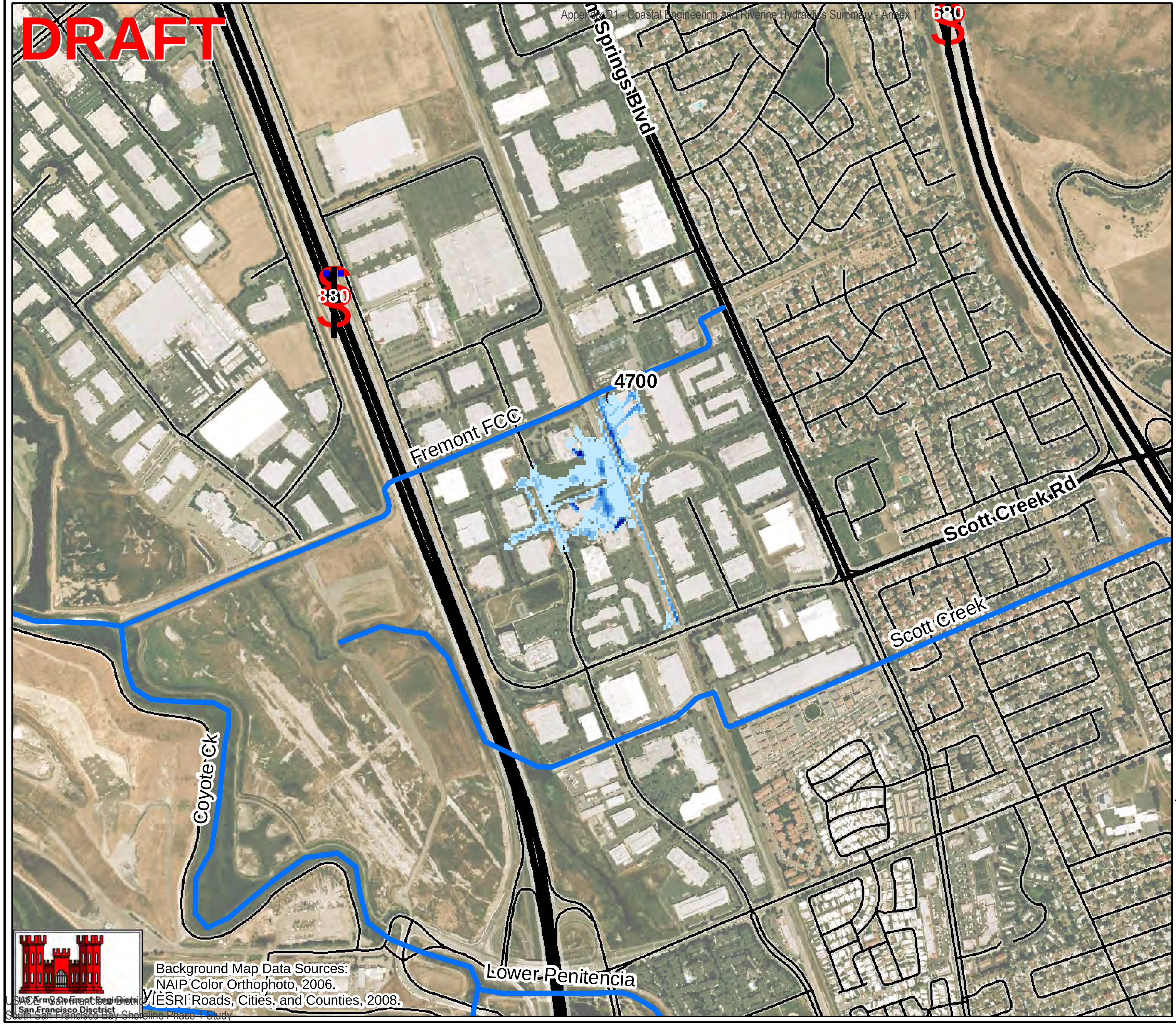
Max Water Depth (ft)

- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 1.5
- 1.5 - 2.0
- 2.0 - 3.0
- 3.0 - 9.0

0.2% Exceedance Probability Fluvial Flood Inundation Map

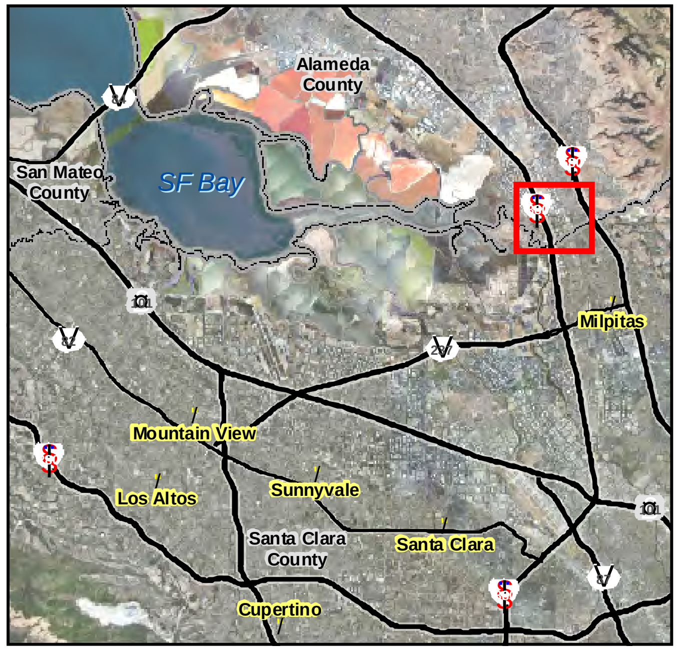
Coyote Creek
Plate 50

US Army Corps of Engineers
San Francisco District



Background Map Data Sources:
NAIP Color Orthophoto, 2006.
ESRI Roads, Cities, and Counties, 2008.

Regional Reference Map



Scale - 1:10,000

Scale - 1 in = 833.3 feet

0 650 1,300 2,600 Feet

State Plane CA Zone III	horz. datum: NAD 83 vert. datum: NAVD 88	horz. units: feet 18 Jun 2010

Legend

(Breakout Locations (Stations Labeled)

Stream Centerline

Roads

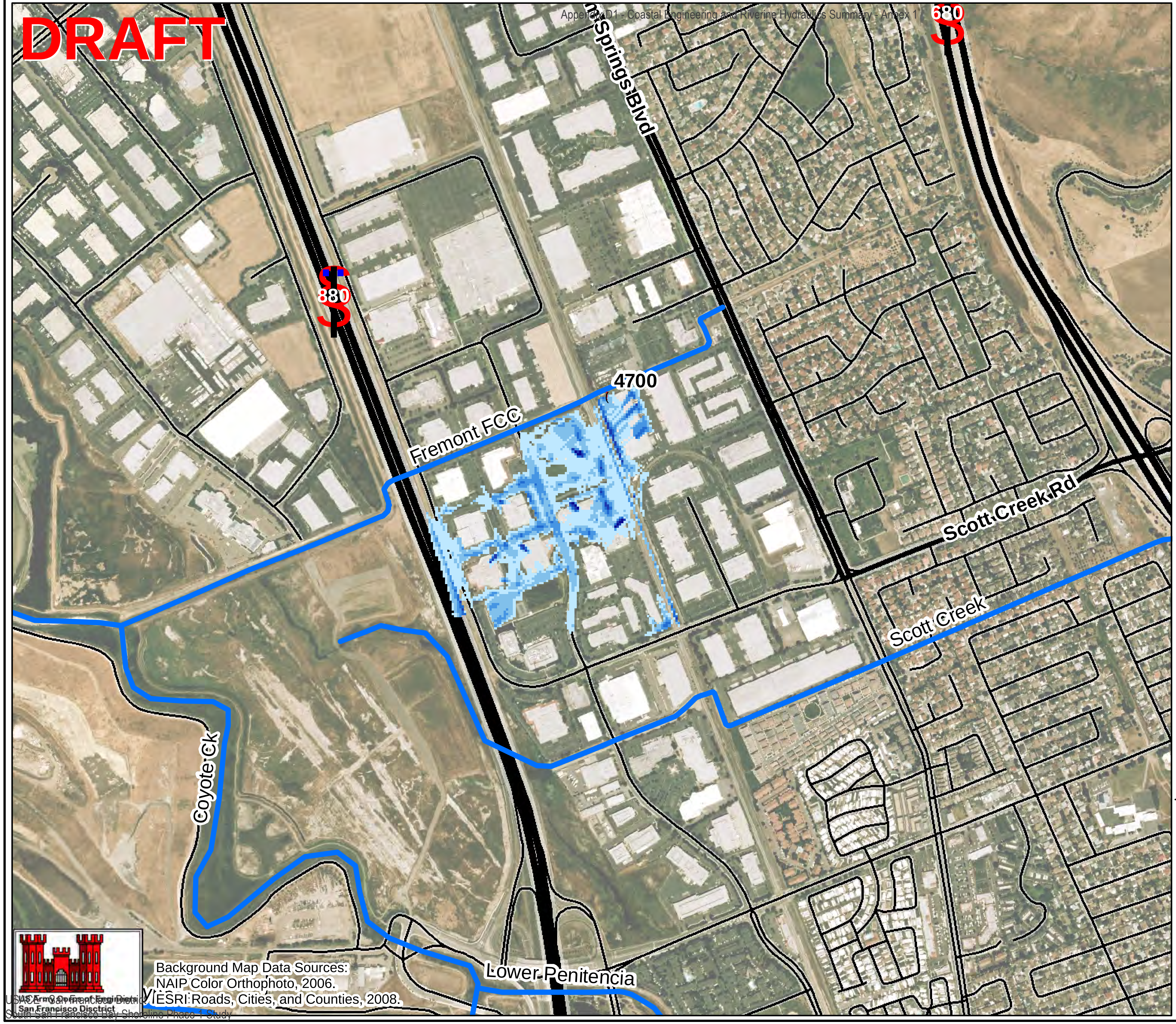
Max Water Depth (ft)

0.0 - 0.5
0.5 - 1.0
1.0 - 1.5
1.5 - 2.0
2.0 - 3.0
3.0 - 9.0

1% Exceedance Probability Fluvial Flood Inundation Map

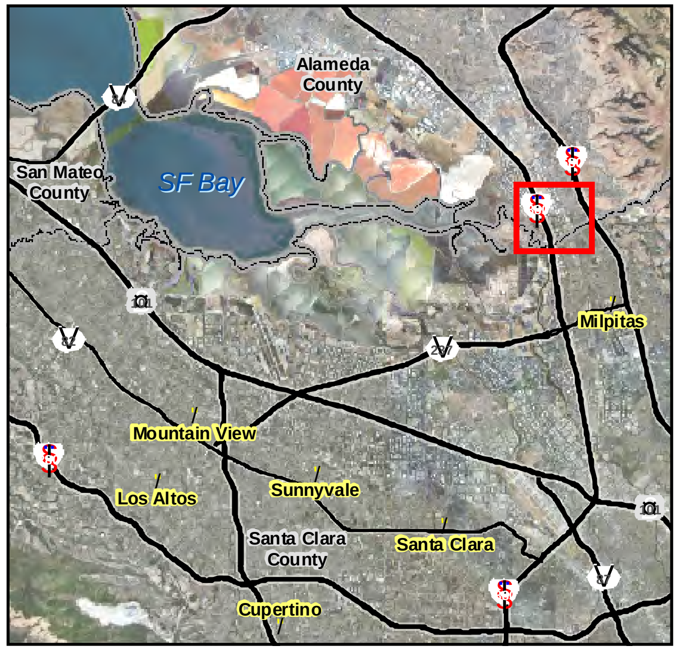
Fremont Creek
Plate 51

US Army Corps of Engineers
San Francisco District



Background Map Data Sources:
NAIP Color Orthophoto, 2006.
ESRI Roads, Cities, and Counties, 2008.

Regional Reference Map



Scale - 1:10,000

Scale - 1 in = 833.3 feet

0 650 1,300 2,600 Feet

State Plane CA Zone III	horz. datum: NAD 83	horz. units: feet
	vert. datum: NAVD 88	18 Jun 2010

Legend

- Breakout Locations (Stations Labeled)
- Stream Centerline
- Roads

Max Water Depth (ft)

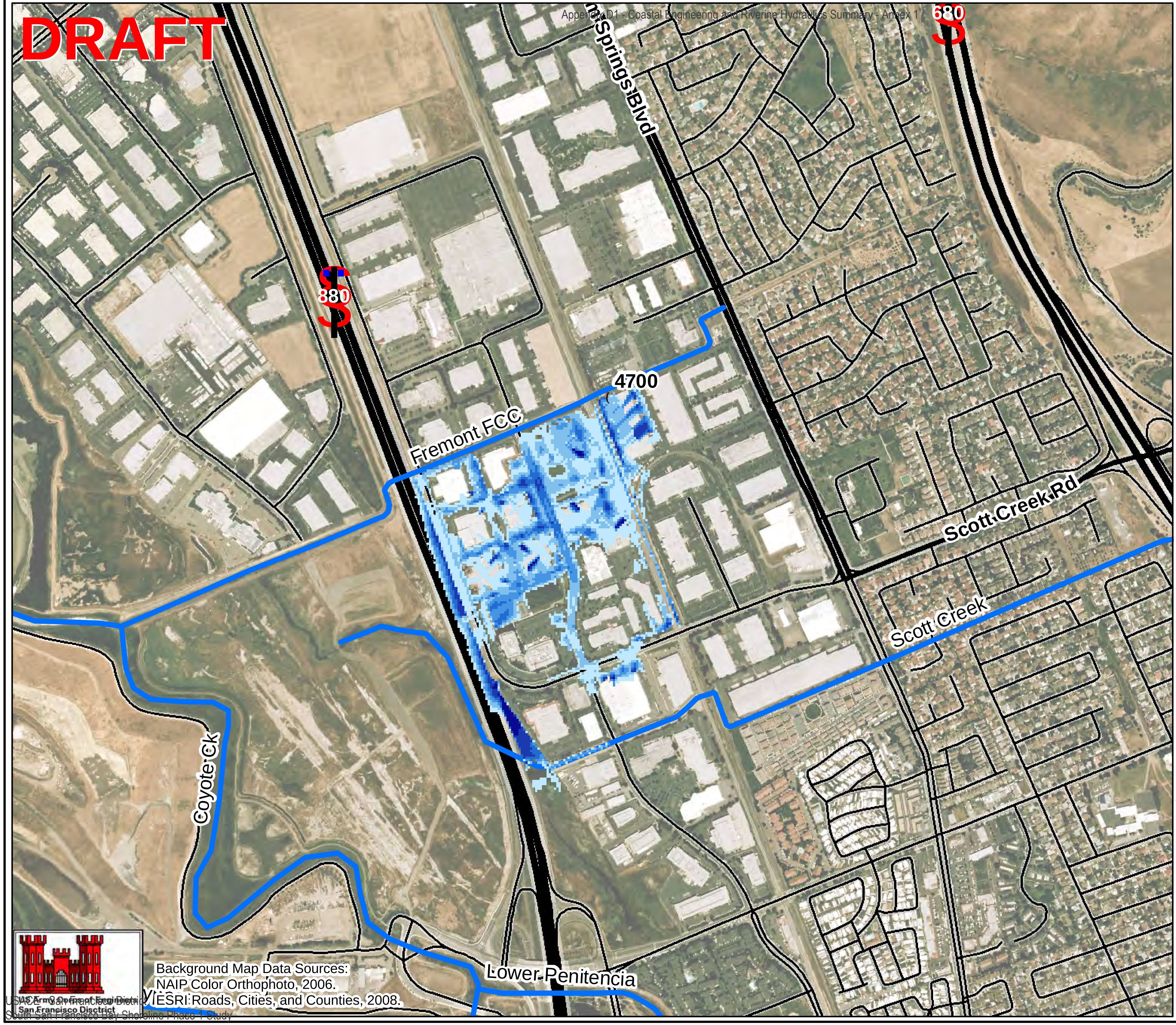
- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 1.5
- 1.5 - 2.0
- 2.0 - 3.0
- 3.0 - 9.0

0.4% Exceedance Probability Fluvial Flood Inundation Map

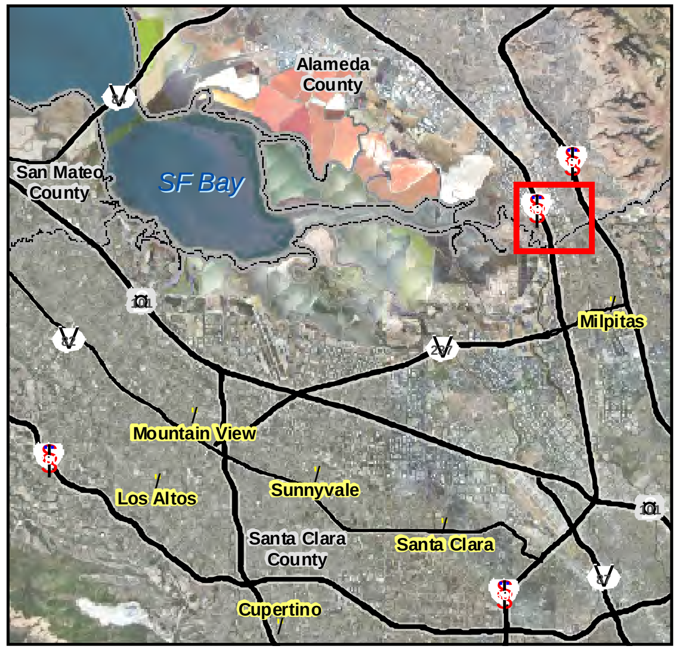
Fremont Creek
Plate 52

US Army Corps of Engineers
San Francisco District

US Army Corps of Engineers San Francisco District
South San Francisco Bay Shoreline Phase I Study



Regional Reference Map



Scale - 1:10,000

Scale - 1 in = 833.3 feet

0 650 1,300 2,600 Feet

State Plane CA Zone III	horz. datum: NAD 83	horz. units: feet
	vert. datum: NAVD 88	18 Jun 2010

Legend

- Breakout Locations (Stations Labeled)
- Stream Centerline
- Roads

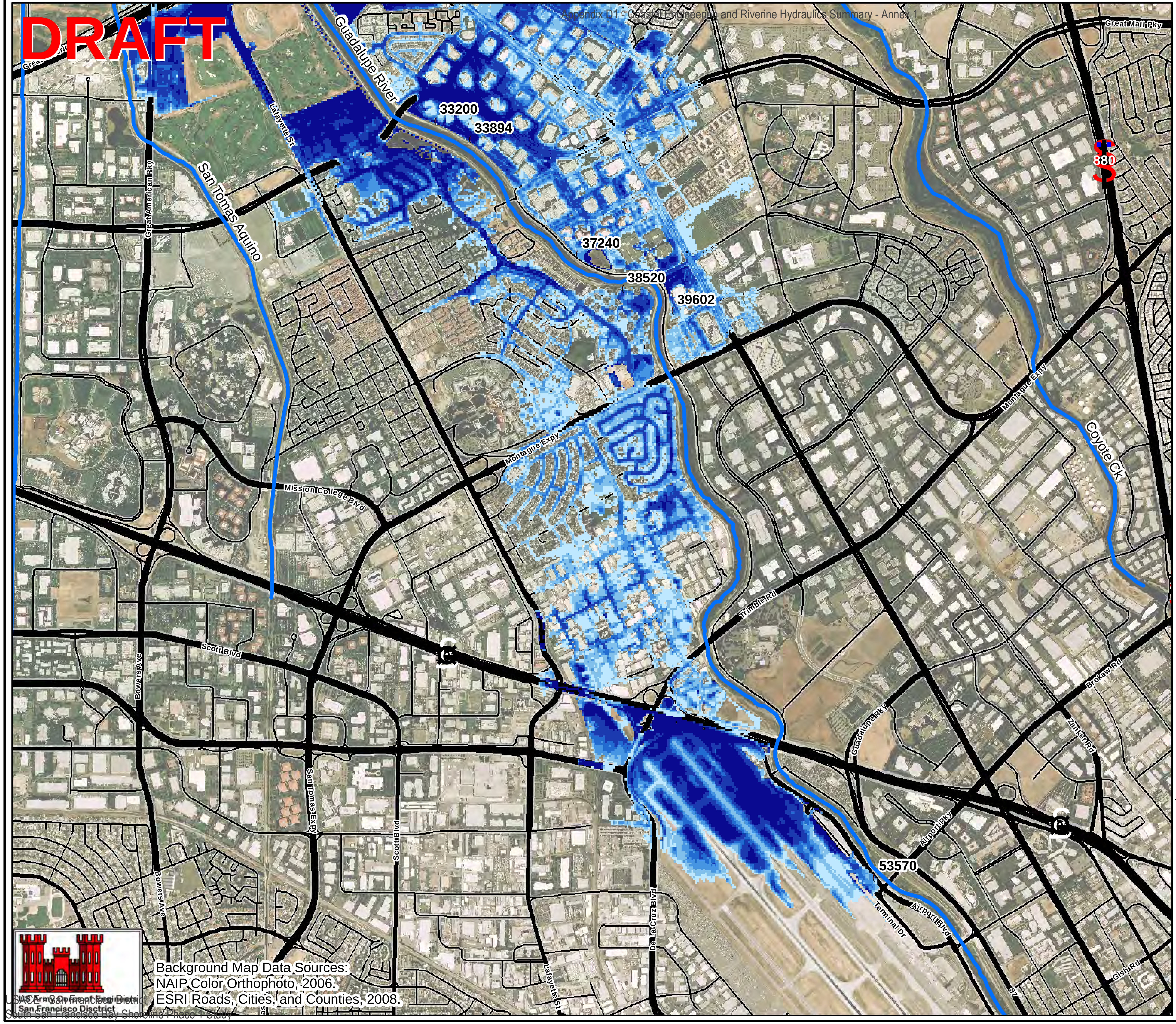
Max Water Depth (ft)

- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 1.5
- 1.5 - 2.0
- 2.0 - 3.0
- 3.0 - 9.0

0.2% Exceedance Probability Fluvial Flood Inundation Map

Fremont Creek
Plate 53

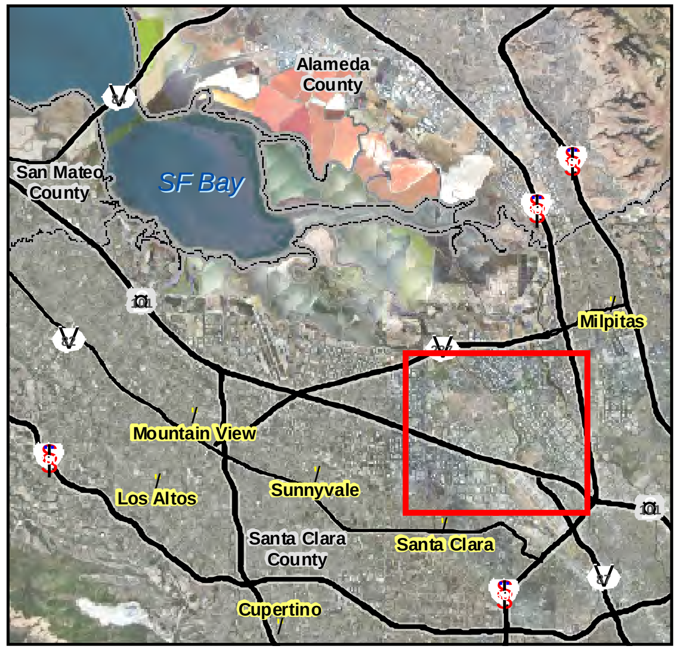
US Army Corps of Engineers
San Francisco District



Background Map Data Sources:
NAIP Color Orthophoto, 2006.
ESRI Roads, Cities, and Counties, 2008.

US Army Corps of Engineers
San Francisco District

Regional Reference Map



Scale - 1:24,000

Scale - 1 in = 2,000 feet

0 1,500 3,000 6,000 Feet

State Plane CA Zone III	horz. datum: NAD 83 vert. datum: NAVD 88	horz. units: feet 18 Jun 2010

Legend

- Breakout Locations (Stations Labeled)
- Stream Centerline
- Roads

Max Water Depth (ft)

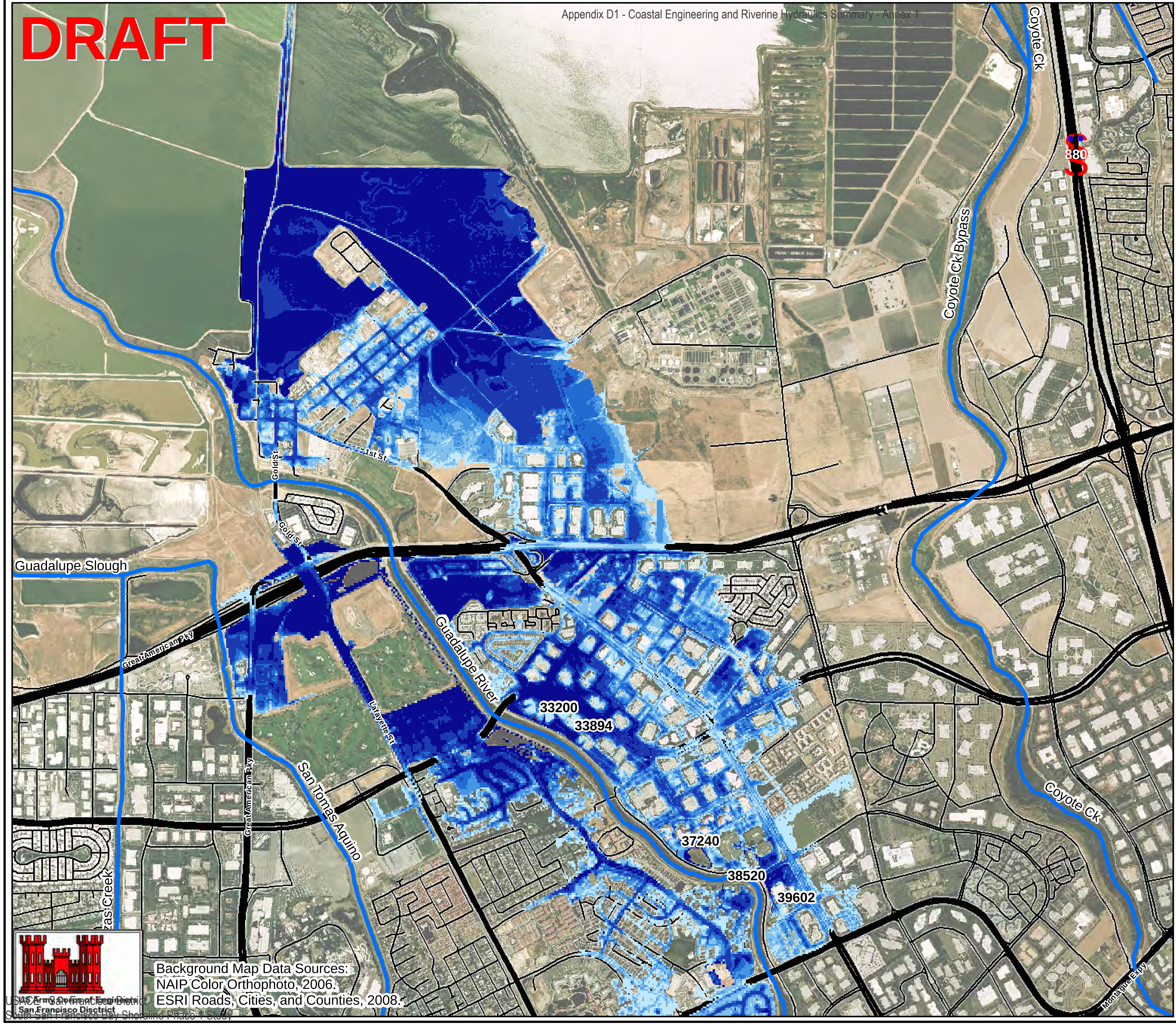
- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 1.5
- 1.5 - 2.0
- 2.0 - 3.0
- 3.0 - 9.0

¶

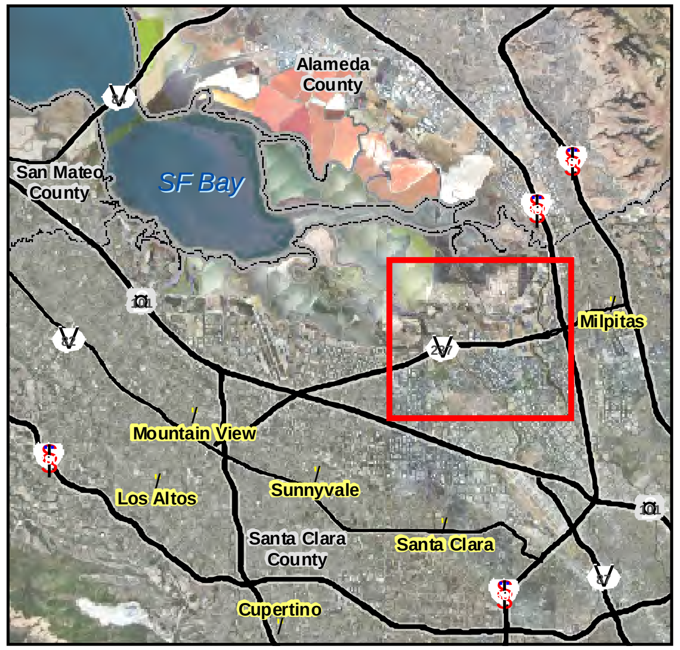
0.2% Exceedance Probability Fluvial Flood Inundation Map

Guadalupe River
Plate 54

US Army Corps of Engineers
San Francisco District



Regional Reference Map



Scale - 1:24,000

Scale - 1 in = 2,000 feet

0 1,500 3,000 6,000 Feet

State Plane CA Zone III	horz. datum: NAD 83	horz. units: feet
	vert. datum: NAVD 88	18 Jun 2010

Legend

(Breakout Locations (Stations Labeled)

Stream Centerline

Roads

Max Water Depth (ft)

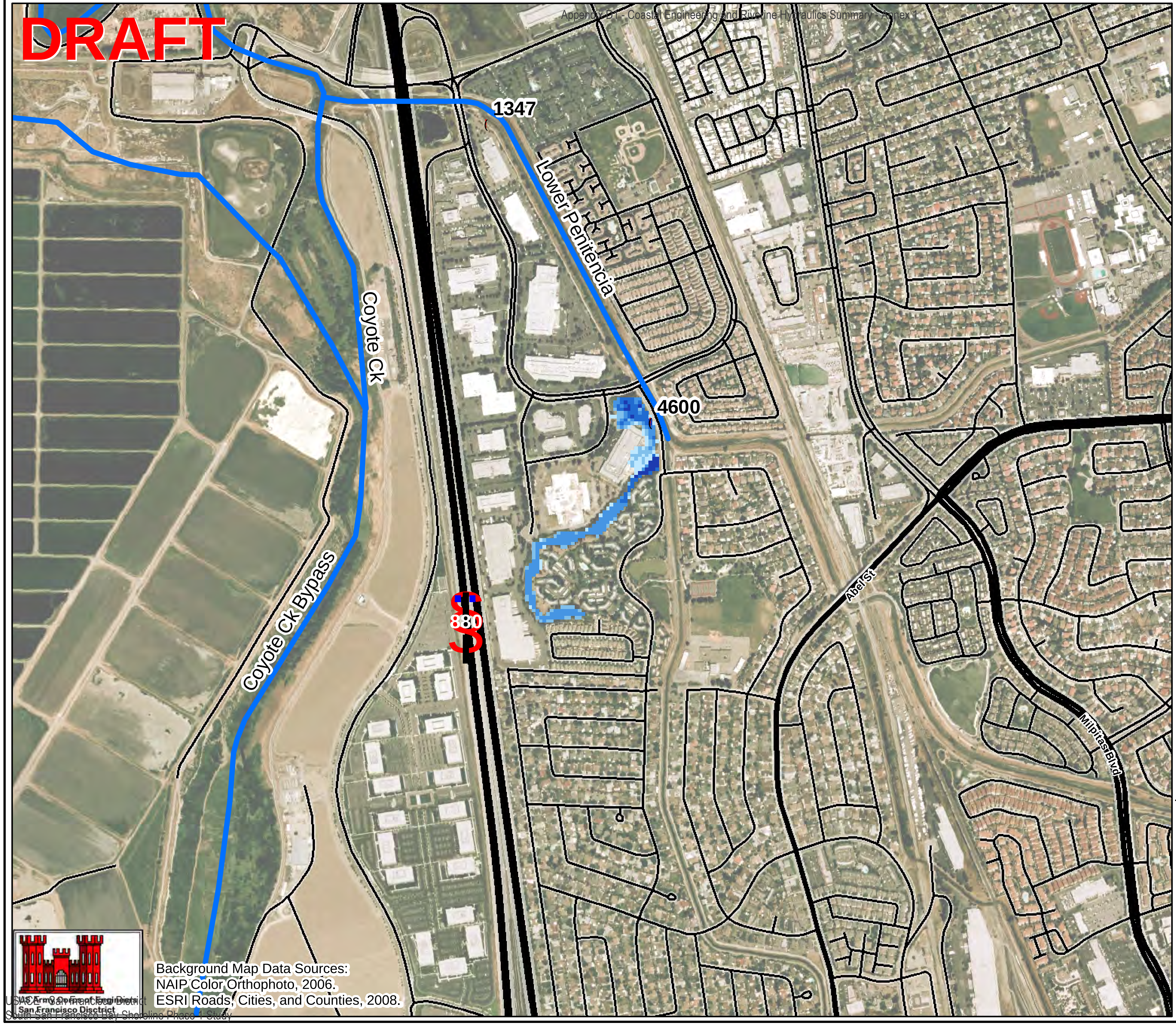
- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 1.5
- 1.5 - 2.0
- 2.0 - 3.0
- 3.0 - 9.0

0.2% Exceedance Probability Fluvial Flood Inundation Map

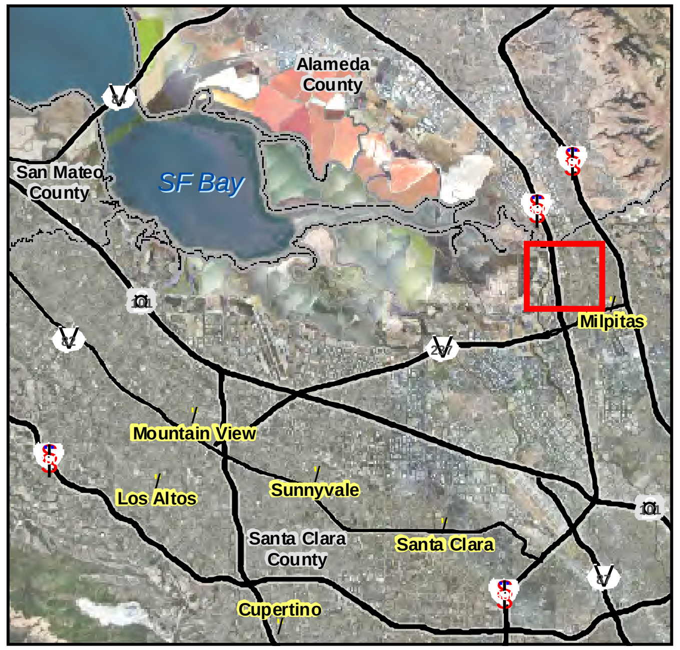
Guadalupe River (cont.)
Plate 55

US Army Corps of Engineers
San Francisco District

US Army Corps of Engineers San Francisco District
South San Francisco Bay Shoreline Phase 1 Study



Regional Reference Map



Scale - 1:10,000

Scale - 1 in = 833.3 feet

0 650 1,300 2,600 Feet

State Plane CA Zone III	horz. datum: NAD 83 vert. datum: NAVD 88	horz. units: feet 18 Jun 2010

Legend

- Breakout Locations (Stations Labeled)
- Stream Centerline
- Roads

Max Water Depth (ft)

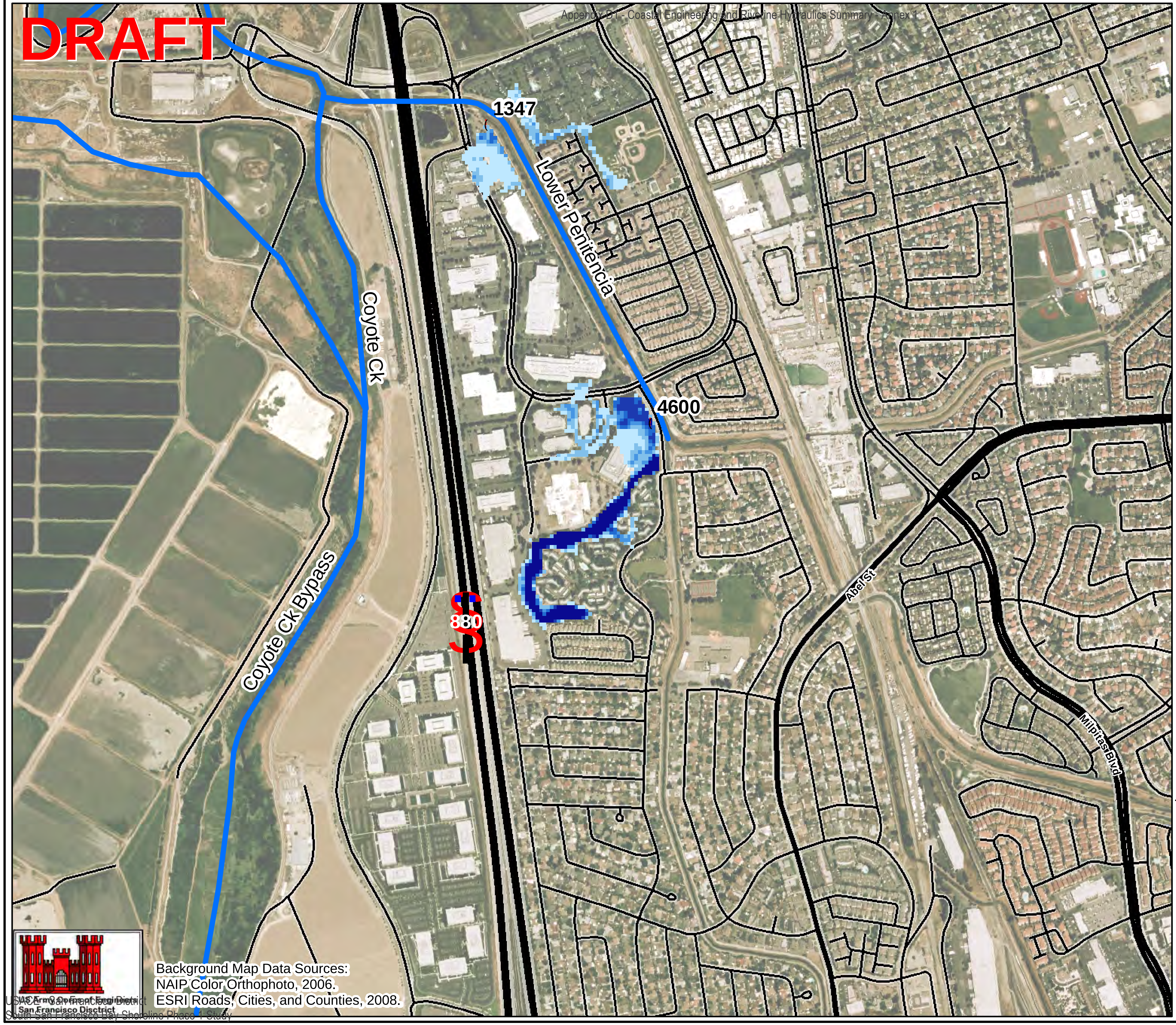
- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 1.5
- 1.5 - 2.0
- 2.0 - 3.0
- 3.0 - 9.0

0.4% Exceedance Probability Fluvial Flood Inundation Map

Lower Penitencia Creek
Plate 56

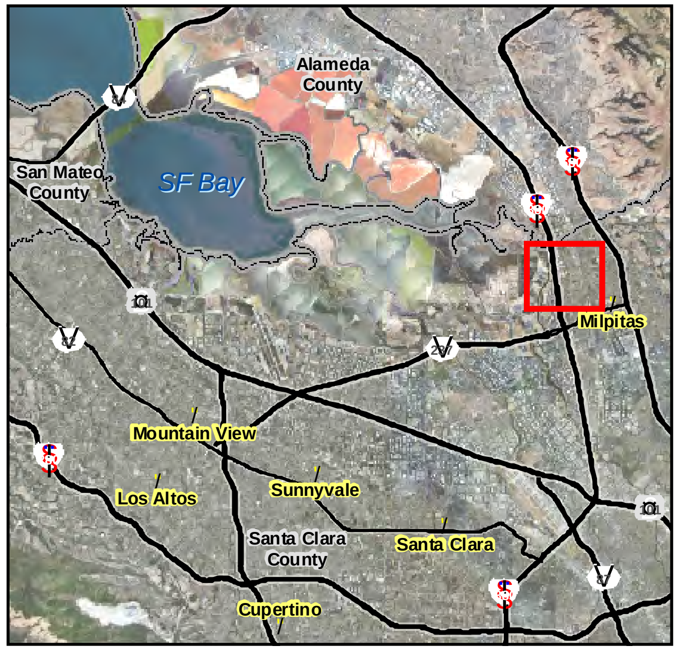
US Army Corps of Engineers
San Francisco District

Background Map Data Sources:
NAIP Color Orthophoto, 2006.
ESRI Roads, Cities, and Counties, 2008.



Background Map Data Sources:
NAIP Color Orthophoto, 2006.
ESRI Roads, Cities, and Counties, 2008.

Regional Reference Map



Scale - 1:10,000
Scale - 1 in = 833.3 feet
0 650 1,300 2,600 Feet

State Plane CA Zone III	horz. datum: NAD 83 vert. datum: NAVD 88	horz. units: feet 18 Jun 2010

Legend

(Breakout Locations (Stations Labeled)
Stream Centerline
Roads
Max Water Depth (ft)
0.0 - 0.5
0.5 - 1.0
1.0 - 1.5
1.5 - 2.0
2.0 - 3.0
3.0 - 9.0

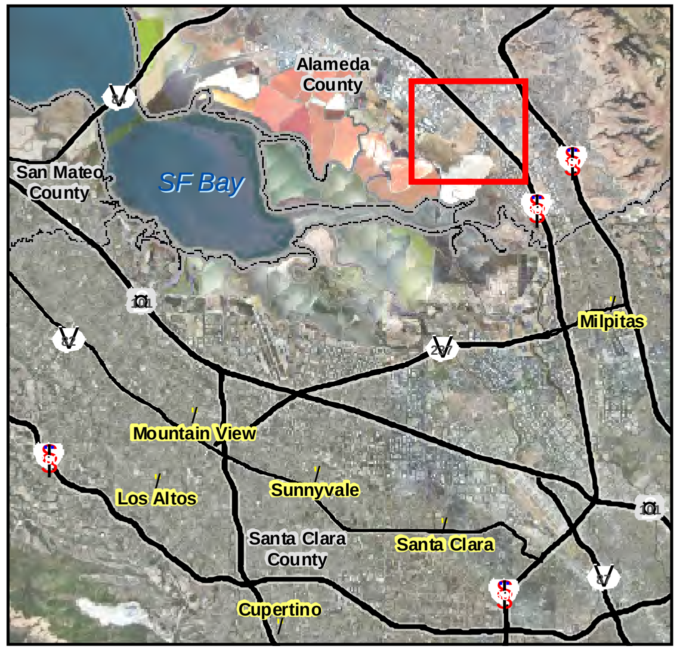
N

0.2% Exceedance Probability Fluvial Flood Inundation Map

Lower Penitencia Creek
Plate 57

US Army Corps of Engineers
San Francisco District

Regional Reference Map



Scale - 1:15,000

Scale - 1 in = 1,250 feet

0 1,000 2,000 4,000 Feet

State Plane CA Zone III	horz. datum: NAD 83 vert. datum: NAVD 88	horz. units: feet 18 Jun 2010

Legend

(Breakout Locations (Stations Labeled)

Stream Centerline

Roads

Max Water Depth (ft)

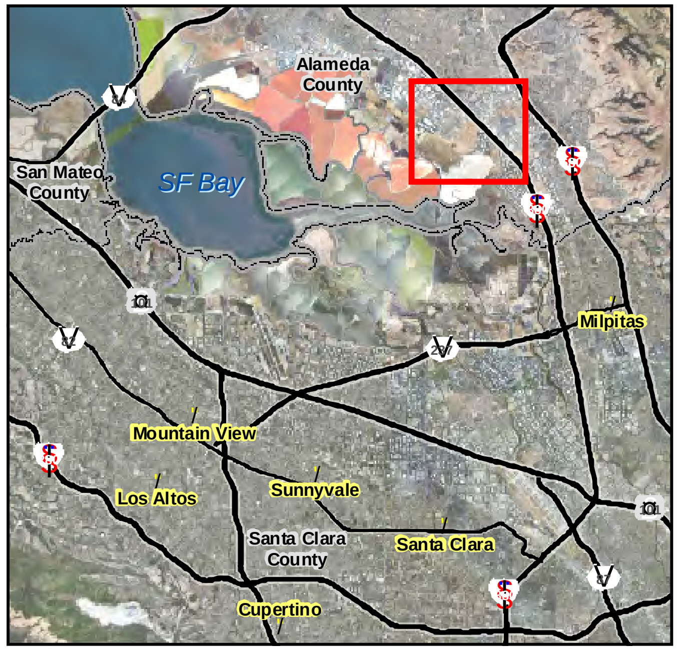
- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 1.5
- 1.5 - 2.0
- 2.0 - 3.0
- 3.0 - 9.0

1% Exceedance Probability Fluvial Flood Inundation Map

Laguna and Agua Caliente Creeks Plate 58

US Army Corps of Engineers
San Francisco District

Regional Reference Map



Scale - 1:15,000

Scale - 1 in = 1,250 feet

0 1,000 2,000 4,000 Feet

State Plane CA Zone III	horz. datum: NAD 83 vert. datum: NAVD 88	horz. units: feet 18 Jun 2010

Legend

- Breakout Locations (Stations Labeled)
- Stream Centerline
- Roads

Max Water Depth (ft)

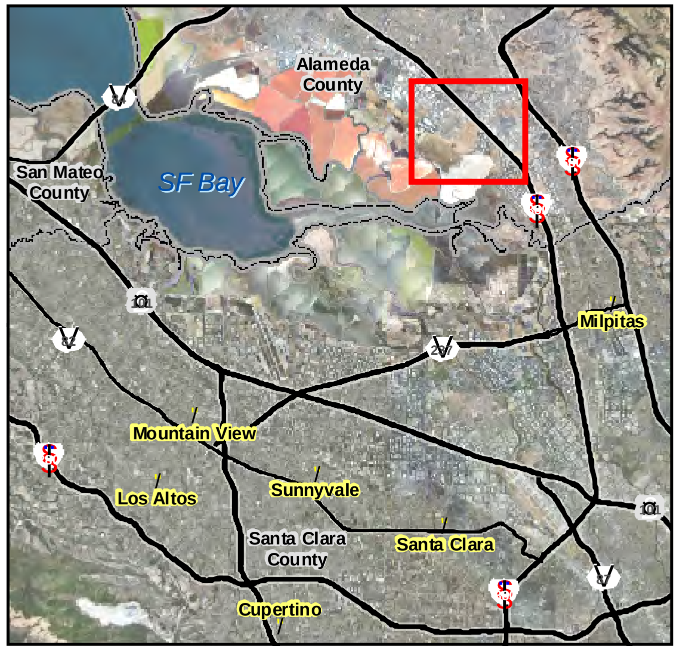
- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 1.5
- 1.5 - 2.0
- 2.0 - 3.0
- 3.0 - 9.0

0.4% Exceedance Probability Fluvial Flood Inundation Map

Laguna and Agua Caliente Creeks Plate 59

US Army Corps of Engineers
San Francisco District

Regional Reference Map



Scale - 1:15,000

Scale - 1 in = 1,250 feet

0 1,000 2,000 4,000 Feet

State Plane CA Zone III	horz. datum: NAD 83 vert. datum: NAVD 88	horz. units: feet 18 June 2010

Legend

- Breakout Locations (Stations Labeled)
- Stream Centerline
- Roads

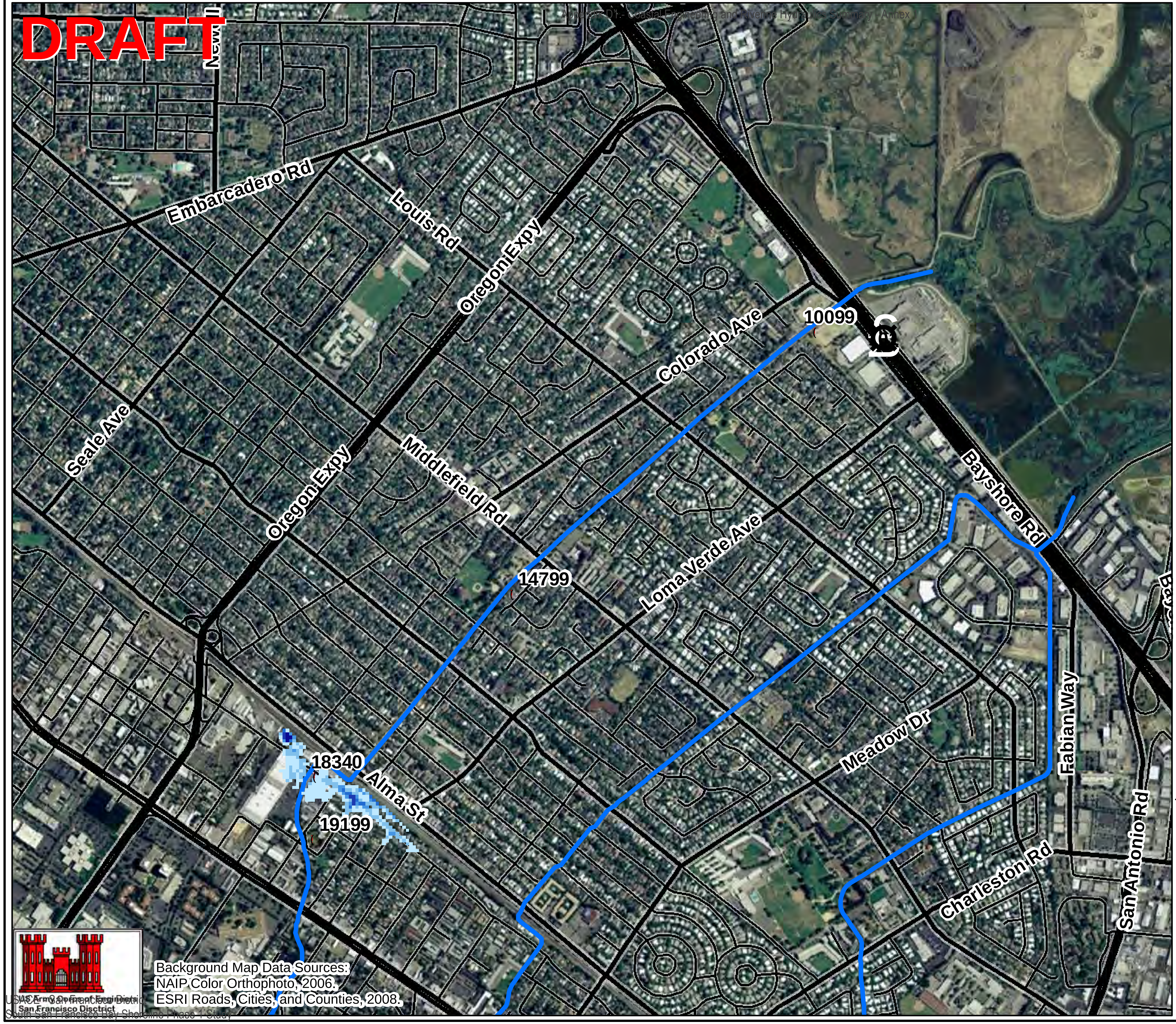
Max Water Depth (ft)

- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 1.5
- 1.5 - 2.0
- 2.0 - 3.0
- 3.0 - 9.0

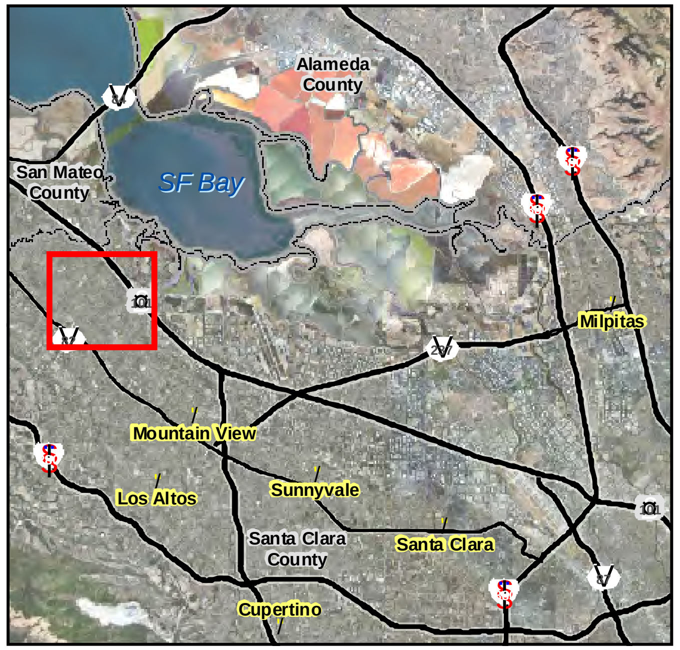
0.2% Exceedance Probability Fluvial Flood Inundation Map

Laguna and Agua Caliente Creeks Plate 60

US Army Corps of Engineers
San Francisco District



Regional Reference Map



Scale - 1:14,000

Scale - 1 in = 1,166.7 feet

0 950 1,900 3,800 Feet

State Plane CA Zone III	horz. datum: NAD 83 vert. datum: NAVD 88	horz. units: feet 18 June 2010

Legend

- Breakout Locations (Stations Labeled)
- Stream Centerline
- Roads

Max Water Depth (ft)

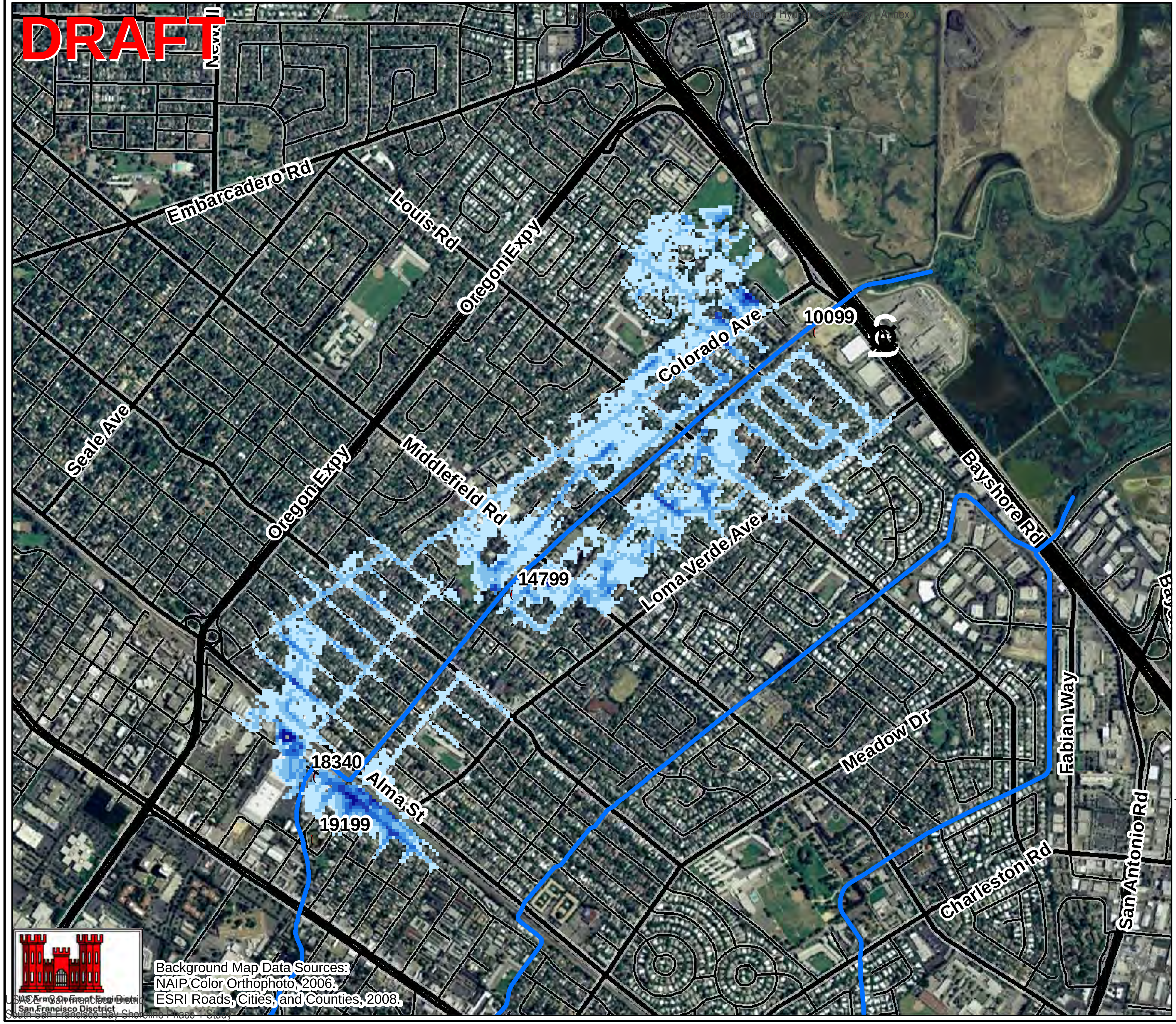
- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 1.5
- 1.5 - 2.0
- 2.0 - 3.0
- 3.0 - 9.0

¶

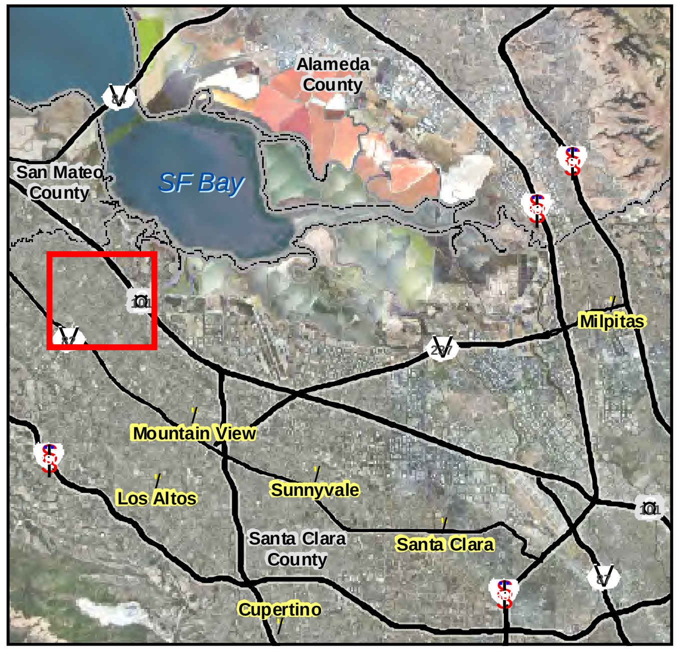
1% Exceedance Probability Fluvial Flood Inundation Map

Matadero Creek
Plate 61

US Army Corps of Engineers
San Francisco District



Regional Reference Map



Scale - 1:14,000

Scale - 1 in = 1,166.7 feet

0 950 1,900 3,800 Feet

State Plane CA Zone III	horz. datum: NAD 83 vert. datum: NAVD 88	horz. units: feet 18 June 2010

Legend

Breakout Locations (Stations Labeled)

Stream Centerline

Roads

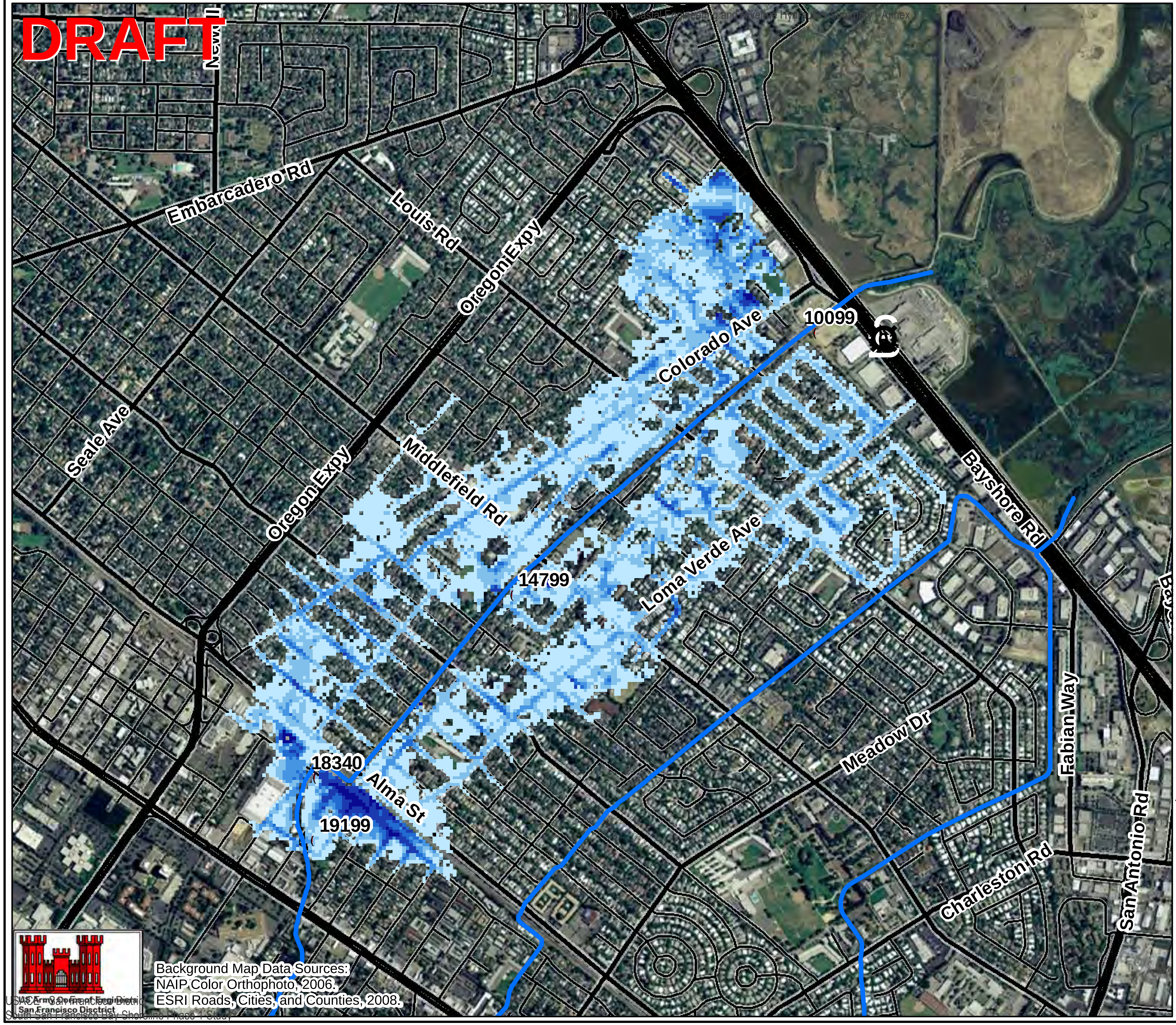
Max Water Depth (ft)

- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 1.5
- 1.5 - 2.0
- 2.0 - 3.0
- 3.0 - 9.0

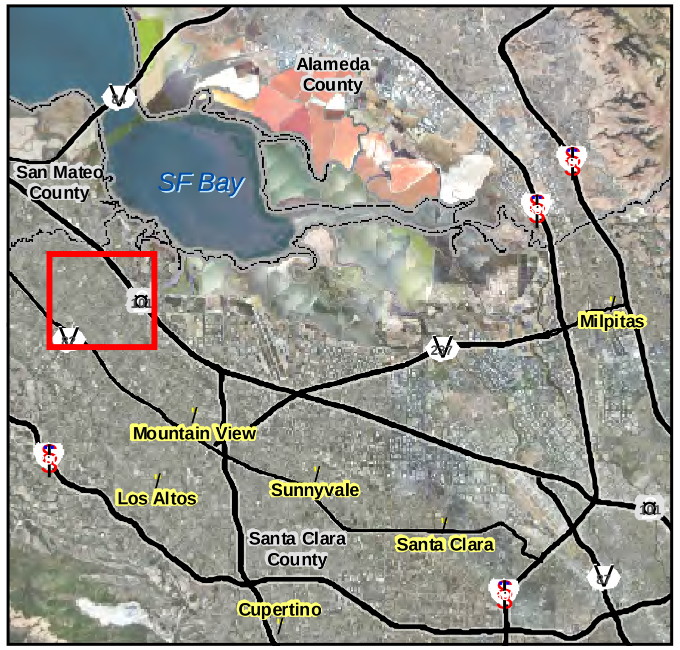
0.4% Exceedance Probability Fluvial Flood Inundation Map

Matadero Creek
Plate 62

US Army Corps of Engineers
San Francisco District



Regional Reference Map



Scale - 1:14,000
Scale - 1 in = 1,166.7 feet
0 950 1,900 3,800 Feet

State Plane CA Zone III	horz. datum: NAD 83 vert. datum: NAVD 88	horz. units: feet 18 June 2010

Legend

(Breakout Locations (Stations Labeled)
Stream Centerline
Roads

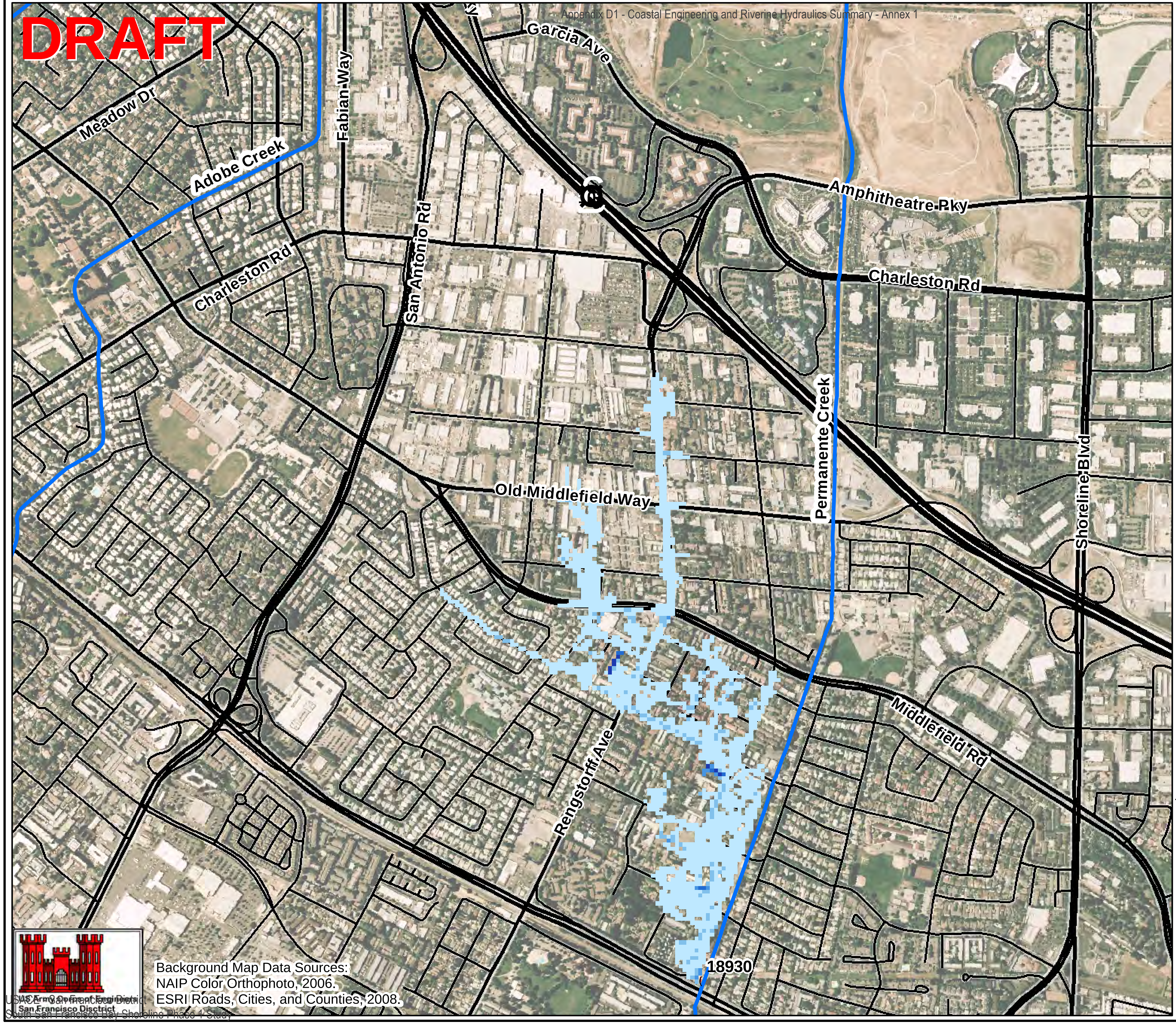
Max Water Depth (ft)

- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 1.5
- 1.5 - 2.0
- 2.0 - 3.0
- 3.0 - 9.0

0.2% Exceedance Probability Fluvial Flood Inundation Map

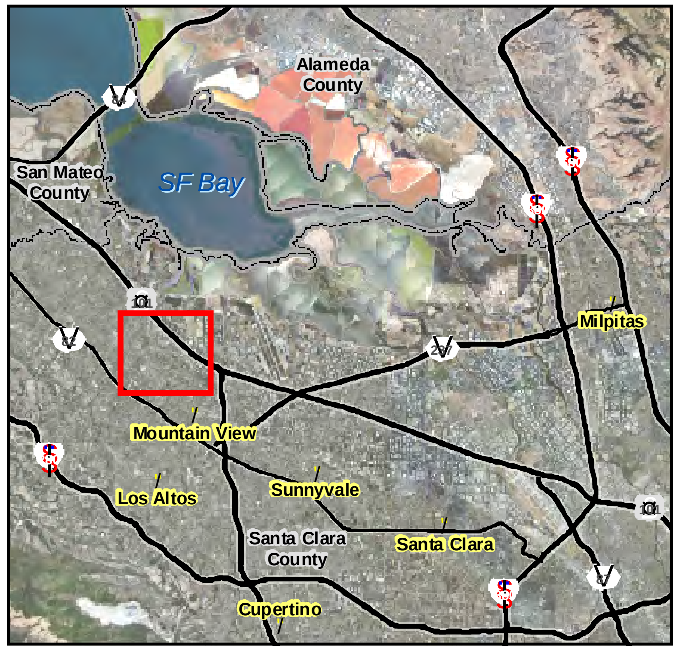
Matadero Creek
Plate 63

US Army Corps of Engineers
San Francisco District



Background Map Data Sources:
NAIP Color Orthophoto, 2006.
ESRI Roads, Cities, and Counties, 2008.

Regional Reference Map



Scale - 1:12,000
Scale - 1 in = 1,000 feet
0 750 1,500 3,000 Feet

State Plane CA Zone III	horz. datum: NAD 83 vert. datum: NAVD 88	horz. units: feet 18 Jun 2010

Legend

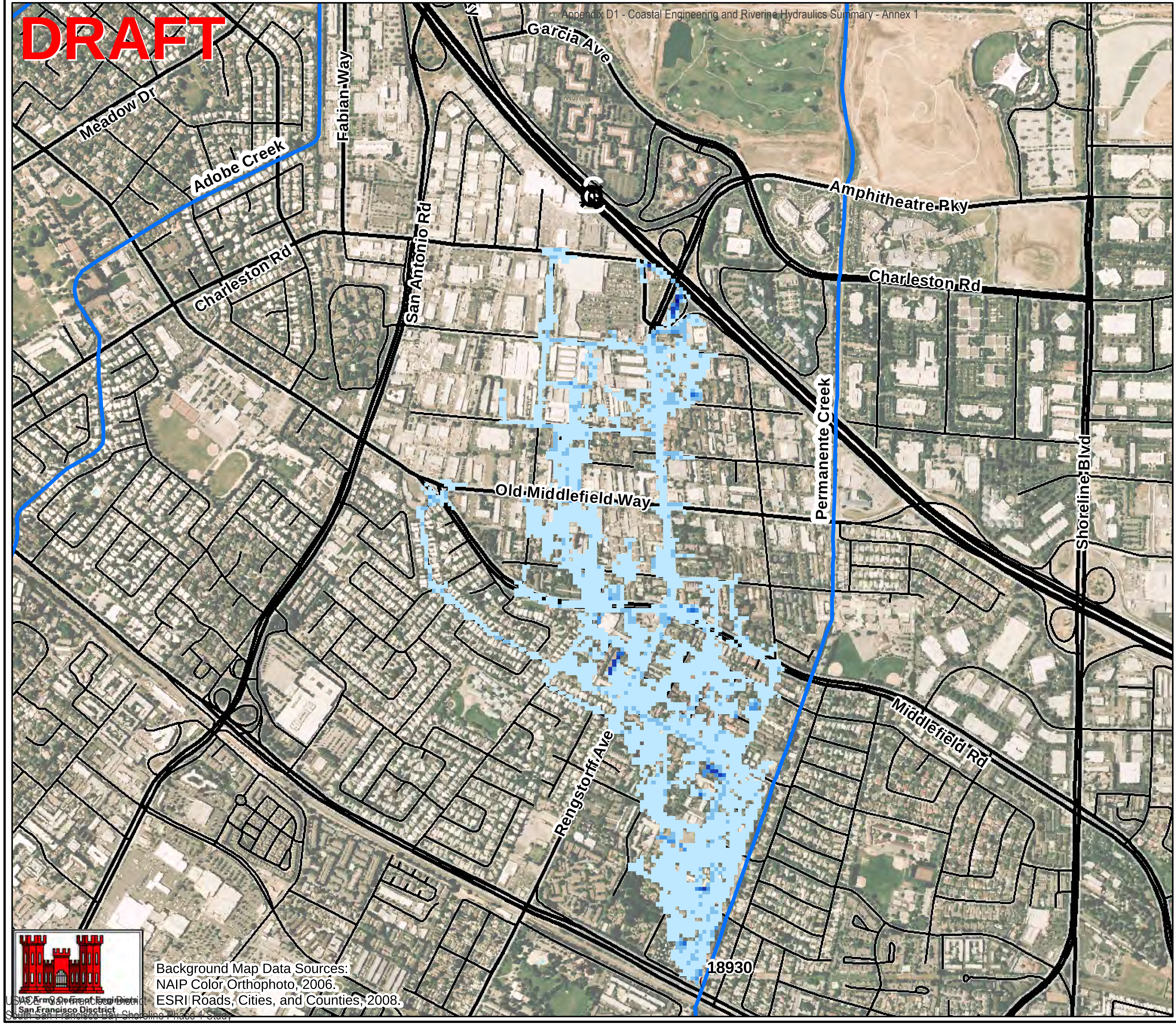
Breakout Locations (Stations Labeled)
Stream Centerline
Roads
Max Water Depth (ft)
0.0 - 0.5
0.5 - 1.0
1.0 - 1.5
1.5 - 2.0
2.0 - 3.0
3.0 - 9.0

1% Exceedance Probability Fluvial Flood Inundation Map

Permanente Creek Plate 64

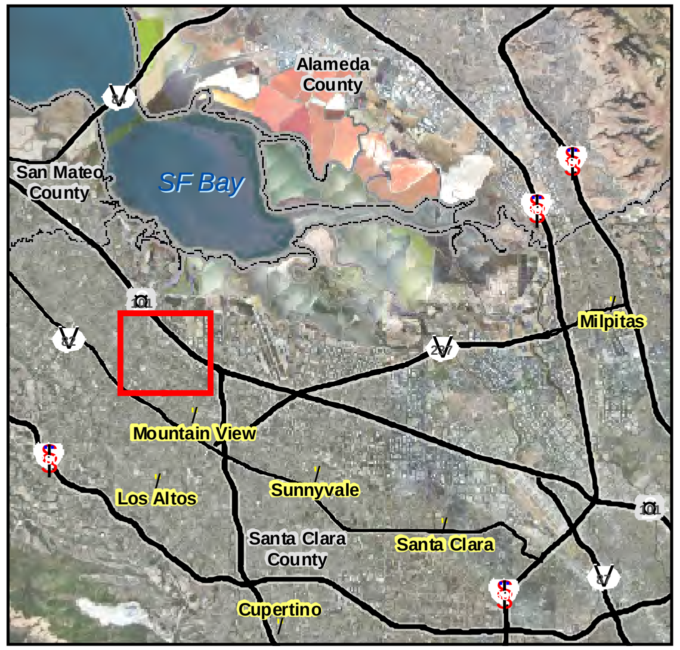
US Army Corps of Engineers San Francisco District

DRAFT



Background Map Data Sources:
NAIP Color Orthophoto, 2006.
ESRI Roads, Cities, and Counties, 2008.

Regional Reference Map



Scale - 1:12,000

Scale - 1 in = 1,000 feet

0 750 1,500 3,000 Feet

State Plane CA Zone III	horz. datum: NAD 83	horz. units: feet
	vert. datum: NAVD 88	18 Jun 2010

Legend

Breakout Locations (Stations Labeled)

Stream Centerline

Roads

Max Water Depth (ft)

0.0 - 0.5

0.5 - 1.0

1.0 - 1.5

1.5 - 2.0

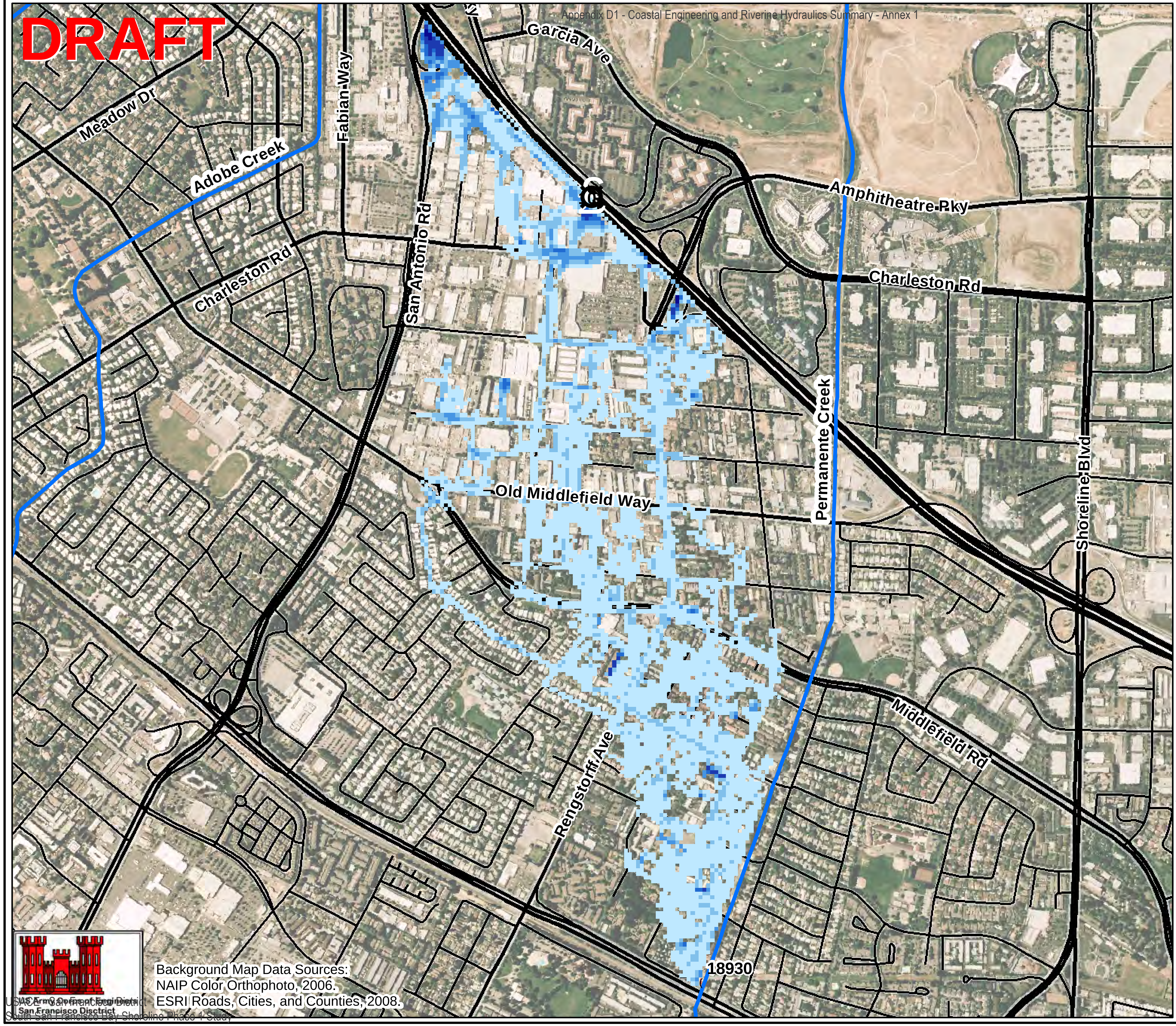
2.0 - 3.0

3.0 - 9.0

0.4% Exceedance Probability Fluvial Flood Inundation Map

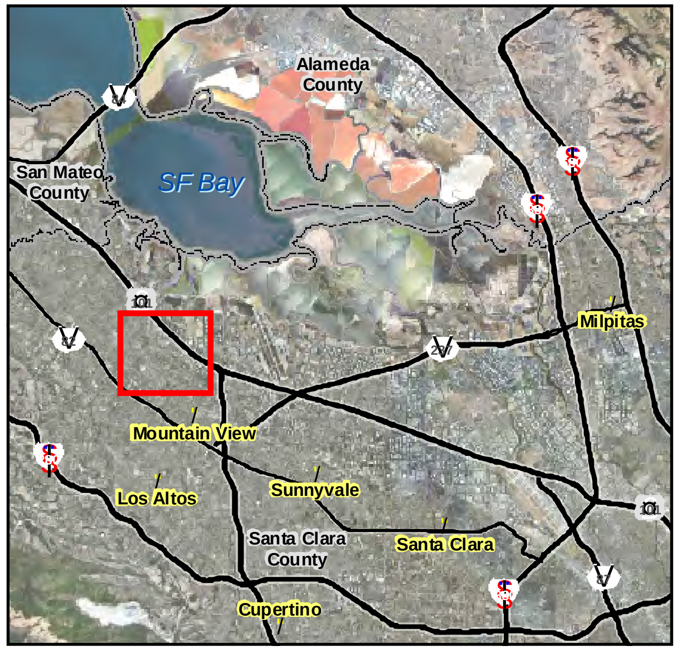
Permanente Creek Plate 65

US Army Corps of Engineers
San Francisco District



Background Map Data Sources:
NAIP Color Orthophoto, 2006.
ESRI Roads, Cities, and Counties, 2008.

Regional Reference Map



Scale - 1:12,000

Scale - 1 in = 1,000 feet

0 750 1,500 3,000 Feet

State Plane CA Zone III	horz. datum: NAD 83 vert. datum: NAVD 88	horz. units: feet 18 Jun 2010

Legend

- Breakout Locations (Stations Labeled)
- Stream Centerline
- Roads

Max Water Depth (ft)

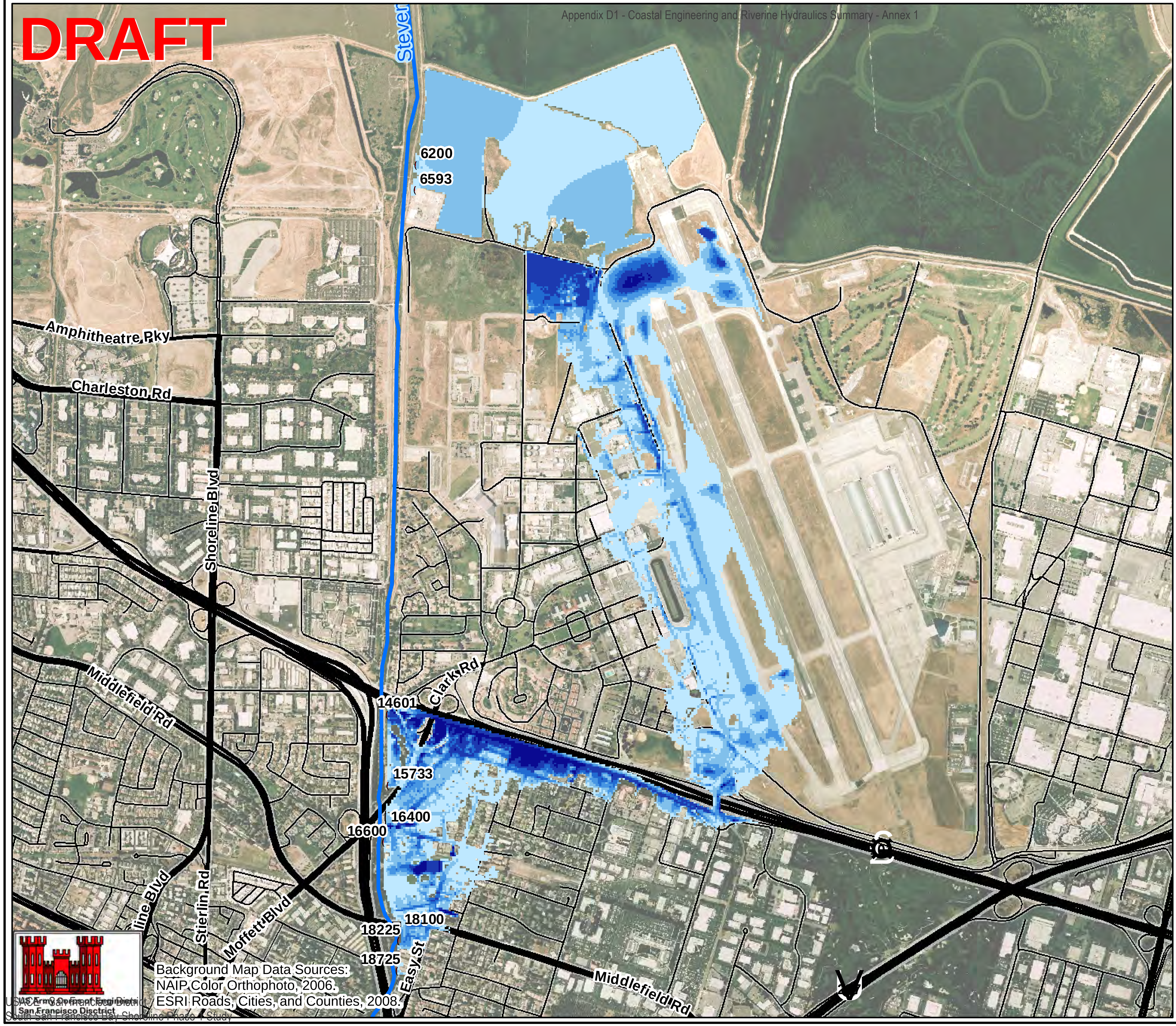
- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 1.5
- 1.5 - 2.0
- 2.0 - 3.0
- 3.0 - 9.0

0.2% Exceedance Probability Fluvial Flood Inundation Map

Permanente Creek
Plate 66

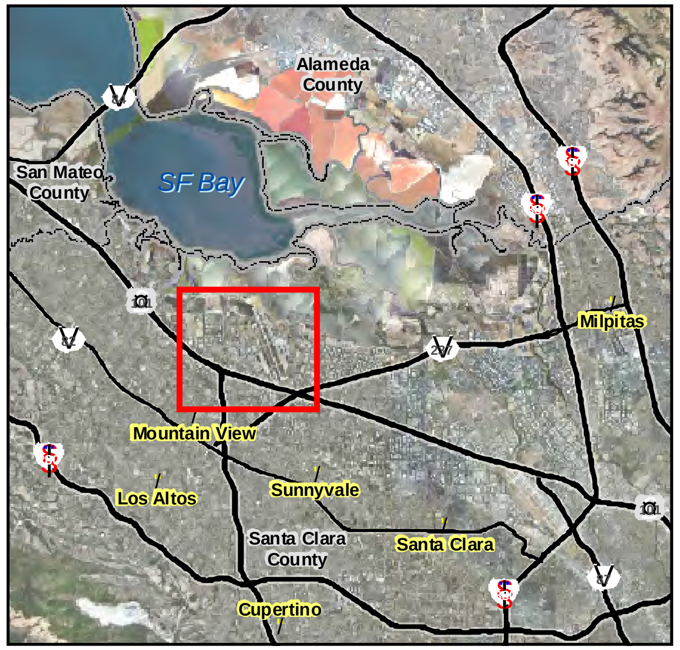
US Army Corps of Engineers
San Francisco District

DRAFT



Background Map Data Sources:
NAIP Color Orthophoto, 2006.
ESRI Roads, Cities, and Counties, 2008.

Regional Reference Map



Scale - 1:18,000
Scale - 1 in = 1,500 feet

0 | 1,200 | 2,400 | 4,800 Feet

State Plane CA Zone III	horz. datum: NAD 83 vert. datum: NAVD 88	horz. units: feet 18 Jun 2010

Legend

- Breakout Locations (Stations Labeled)
- Stream Centerline
- Roads

Max Water Depth (ft)

- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 1.5
- 1.5 - 2.0
- 2.0 - 3.0
- 3.0 - 9.0

1% Exceedance Probability Fluvial Flood Inundation Map

Stevens Creek Plate 67

US Army Corps of Engineers
San Francisco District

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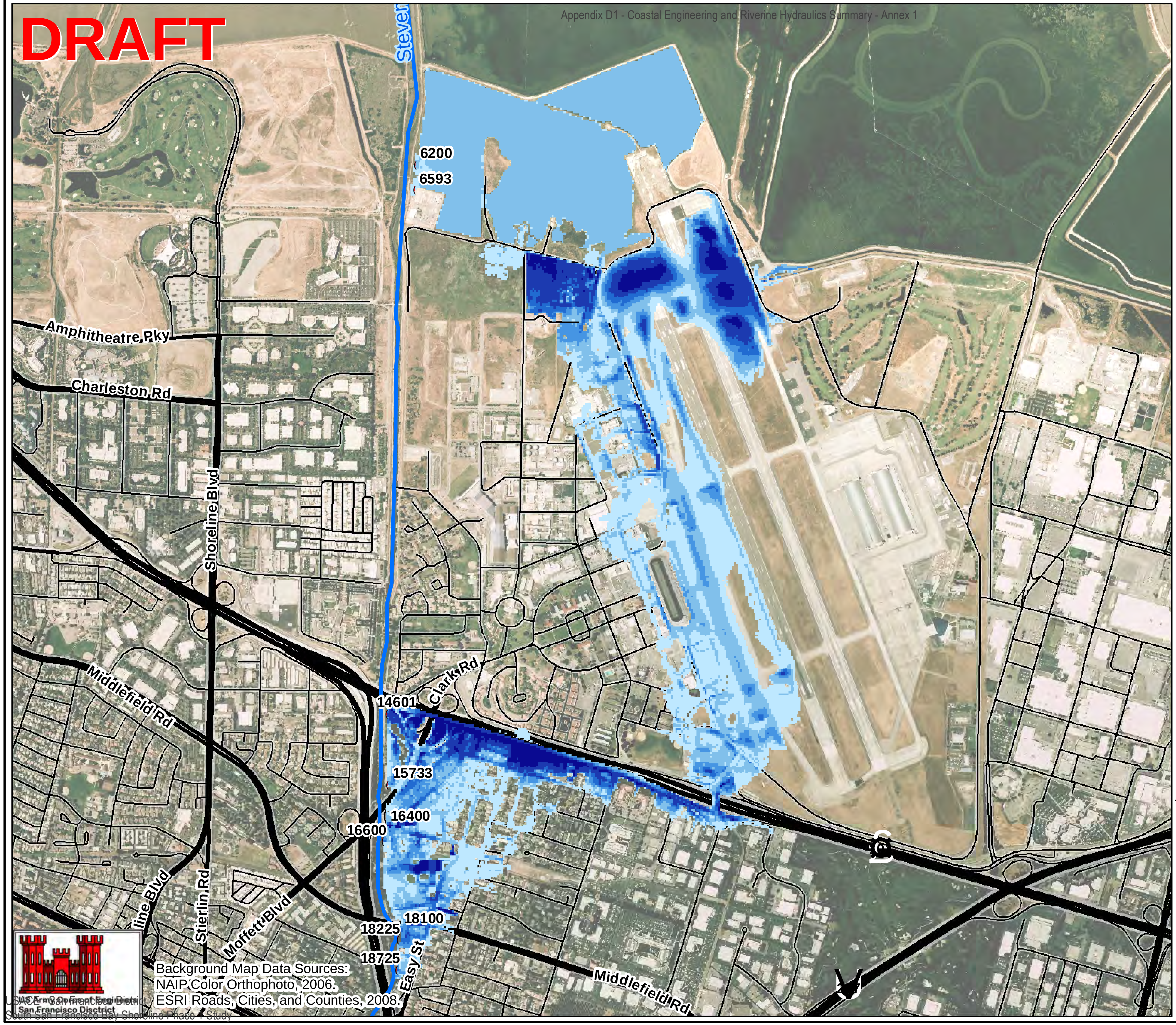
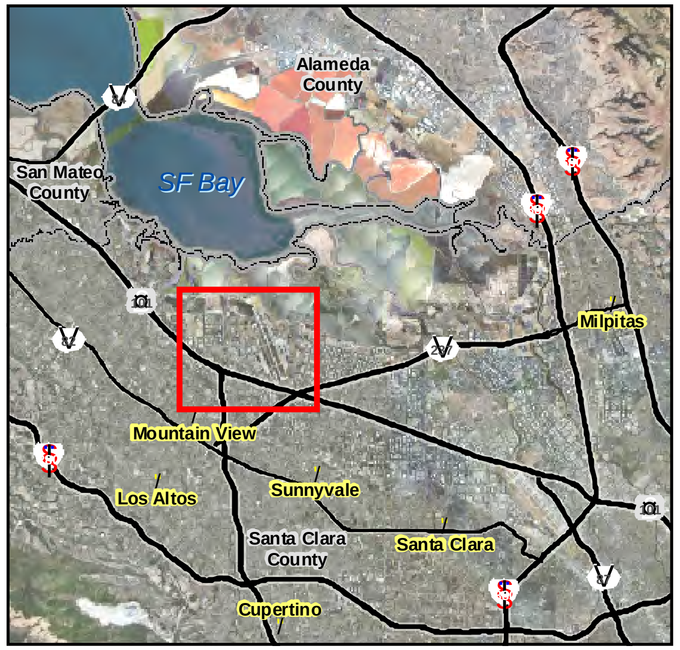


Regional Reference Map



Scale - 1:18,000

Scale - 1 in = 1,500 feet

0 1,200 2,400 4,800 Feet

State Plane CA Zone III	horz. datum: NAD 83 vert. datum: NAVD 88	horz. units: feet 18 Jun 2010

Legend

Breakout Locations (Stations Labeled)

Stream Centerline

Roads

Max Water Depth (ft)

0.0 - 0.5

0.5 - 1.0

1.0 - 1.5

1.5 - 2.0

2.0 - 3.0

3.0 - 9.0

0.4% Exceedance Probability Fluvial Flood Inundation Map

Stevens Creek
Plate 68

US Army Corps of Engineers
San Francisco District

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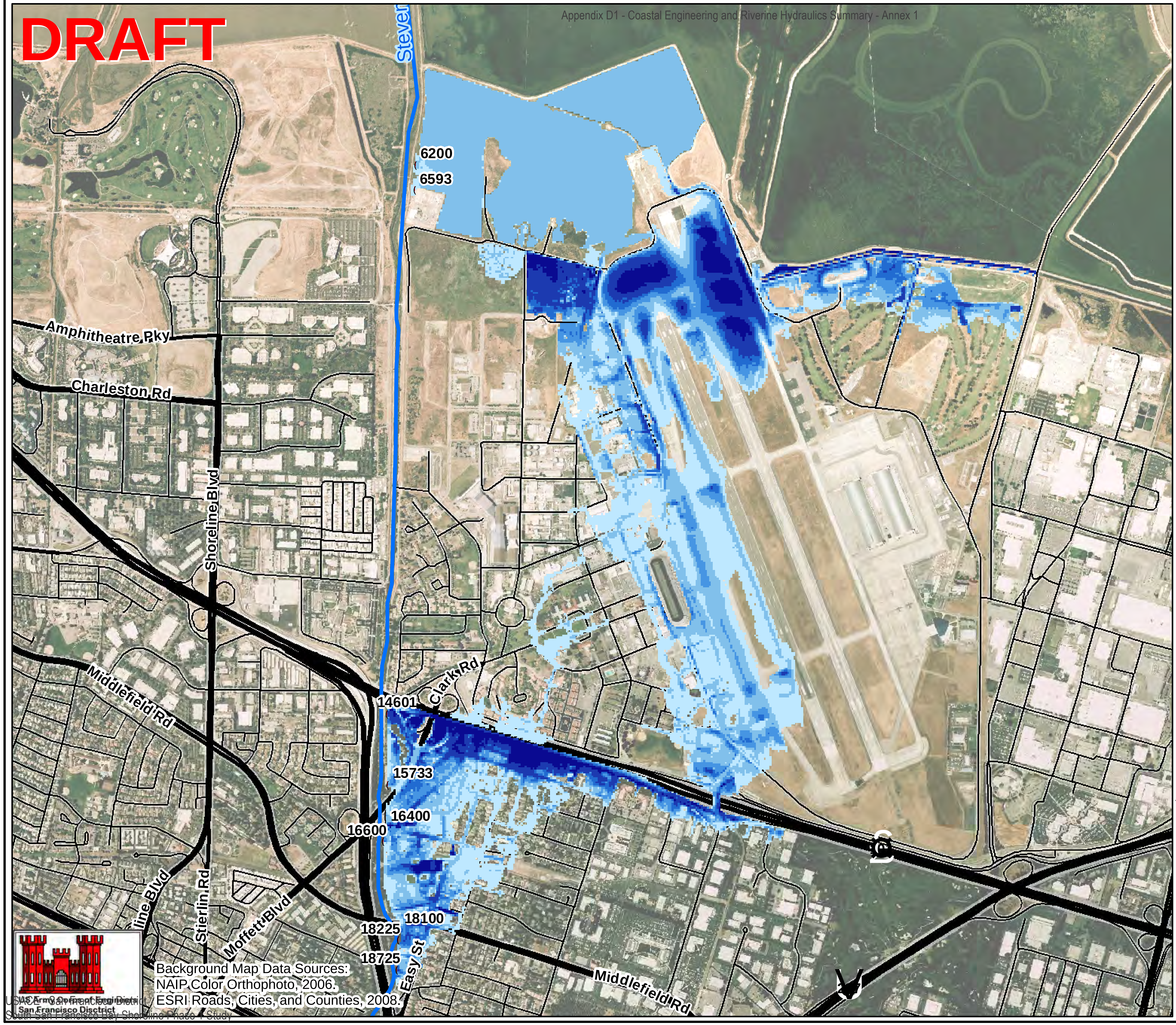
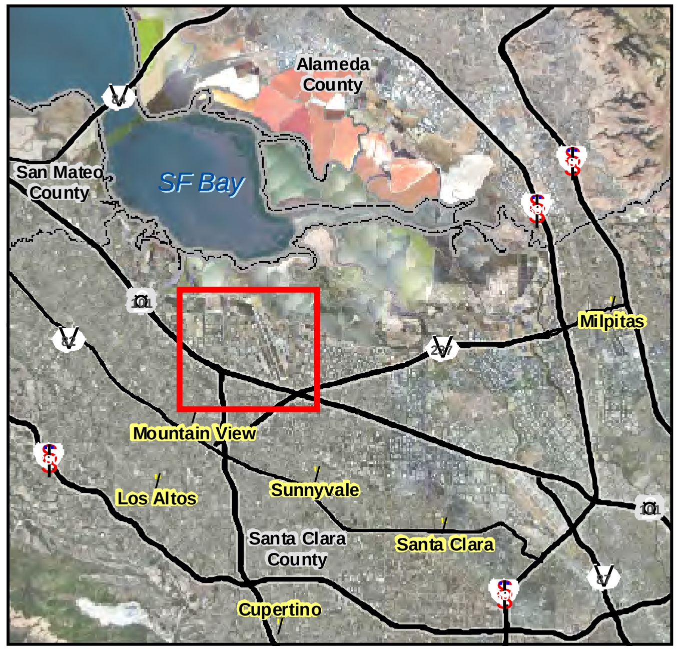


Background Map Data Sources:
NAIP Color Orthophoto, 2006.
ESRI Roads, Cities, and Counties, 2008.

Regional Reference Map



Scale - 1:18,000
Scale - 1 in = 1,500 feet
0 1,200 2,400 4,800 Feet

State Plane CA Zone III	horz. datum: NAD 83 vert. datum: NAVD 88	horz. units: feet 18 Jun 2010

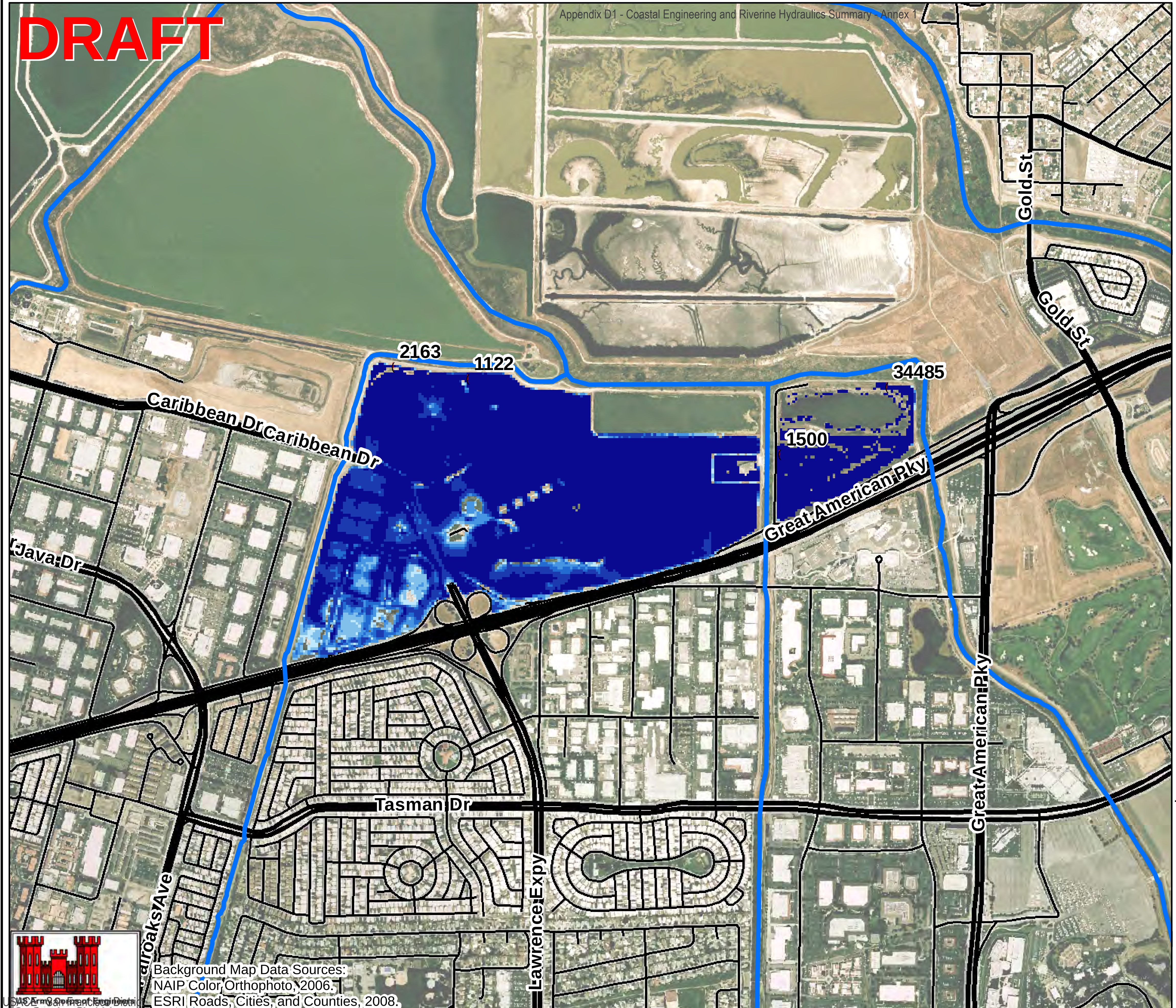
Legend

(Breakout Locations (Stations Labeled)
Stream Centerline
Roads
Max Water Depth (ft)
0.0 - 0.5
0.5 - 1.0
1.0 - 1.5
1.5 - 2.0
2.0 - 3.0
3.0 - 9.0

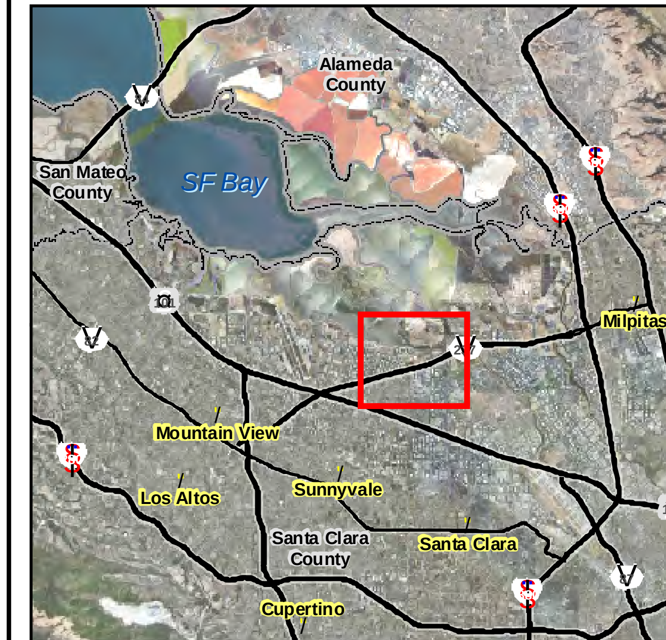
0.2% Exceedance Probability Fluvial Flood Inundation Map

Stevens Creek
Plate 69

US Army Corps of Engineers
San Francisco District



Regional Reference Map



Scale - 1:14,000

Scale - 1 in = 1,166.7 feet

0 875 1,750 3,500 Feet

State Plane CA Zone III	horz. datum: NAD 83 vert. datum: NAVD 88	horz. units: feet 18 Jun 2010

Legend

- Breakout Locations (Stations Labeled)
- Stream Centerline
- Roads

Max Water Depth (ft)

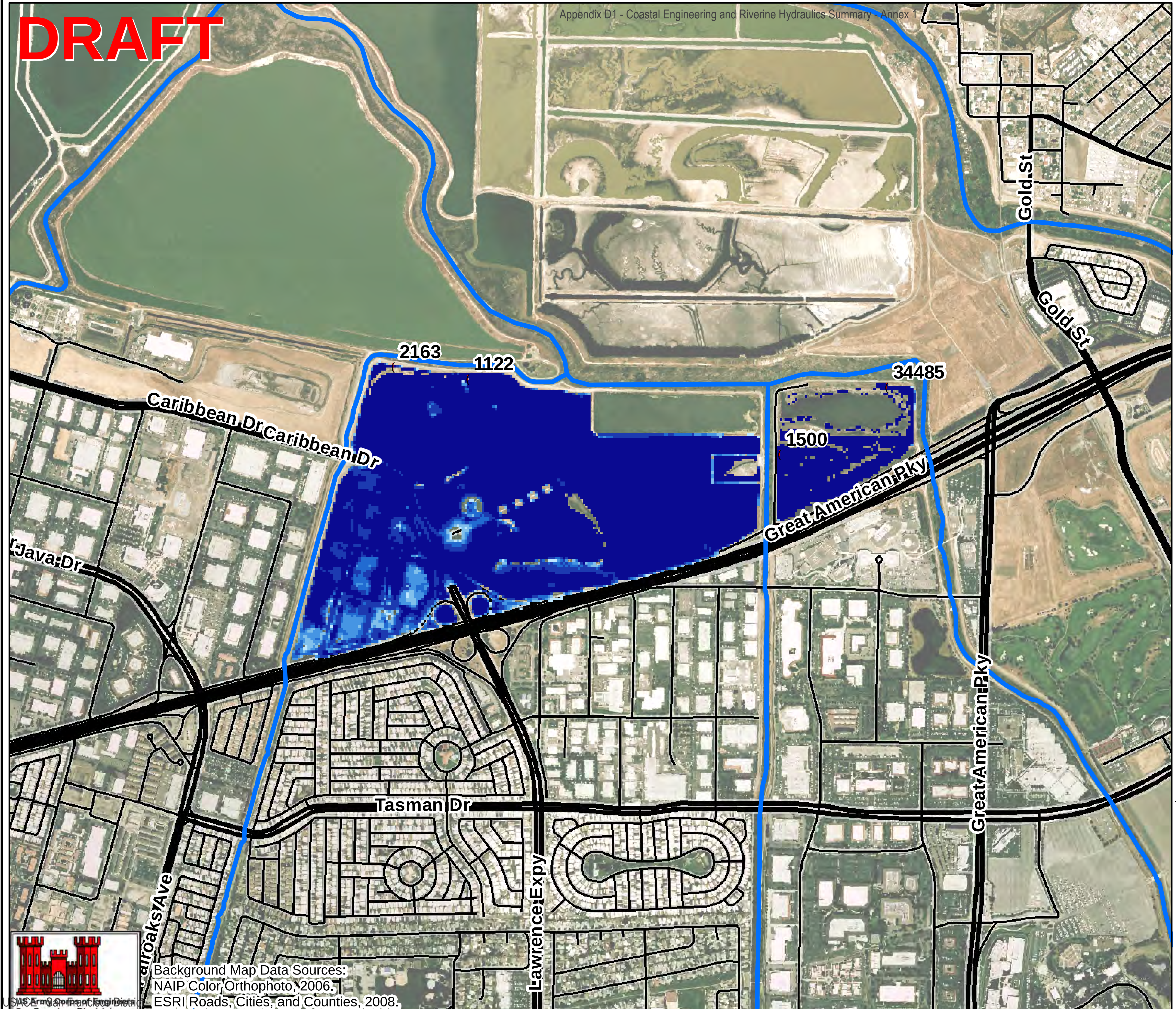
- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 1.5
- 1.5 - 2.0
- 2.0 - 3.0
- 3.0 - 9.0

1% Exceedance Probability Fluvial Flood Inundation Map

Sunnyvale
Plate 70

US Army Corps of Engineers
San Francisco District

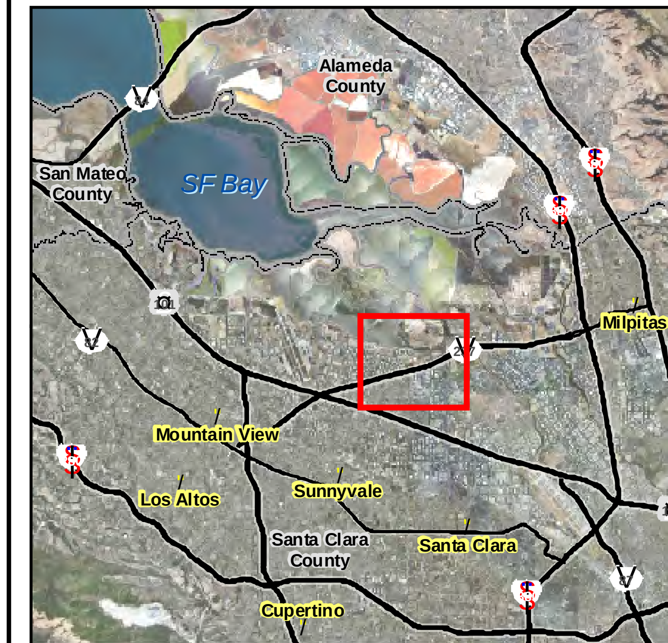
DRAFT



Background Map Data Sources:
NAIP Color Orthophoto, 2006.
ESRI Roads, Cities, and Counties, 2008.

USACE - San Francisco District
South San Francisco Bay Shoreline Phase 1 Study

Regional Reference Map



Scale - 1:14,000
Scale - 1 in = 1,166.7 feet
0 875 1,750 3,500 Feet

State Plane CA Zone III	horz. datum: NAD 83 vert. datum: NAVD 88	horz. units: feet 18 Jun 2010

Legend

- Breakout Locations (Stations Labeled)
- Stream Centerline
- Roads

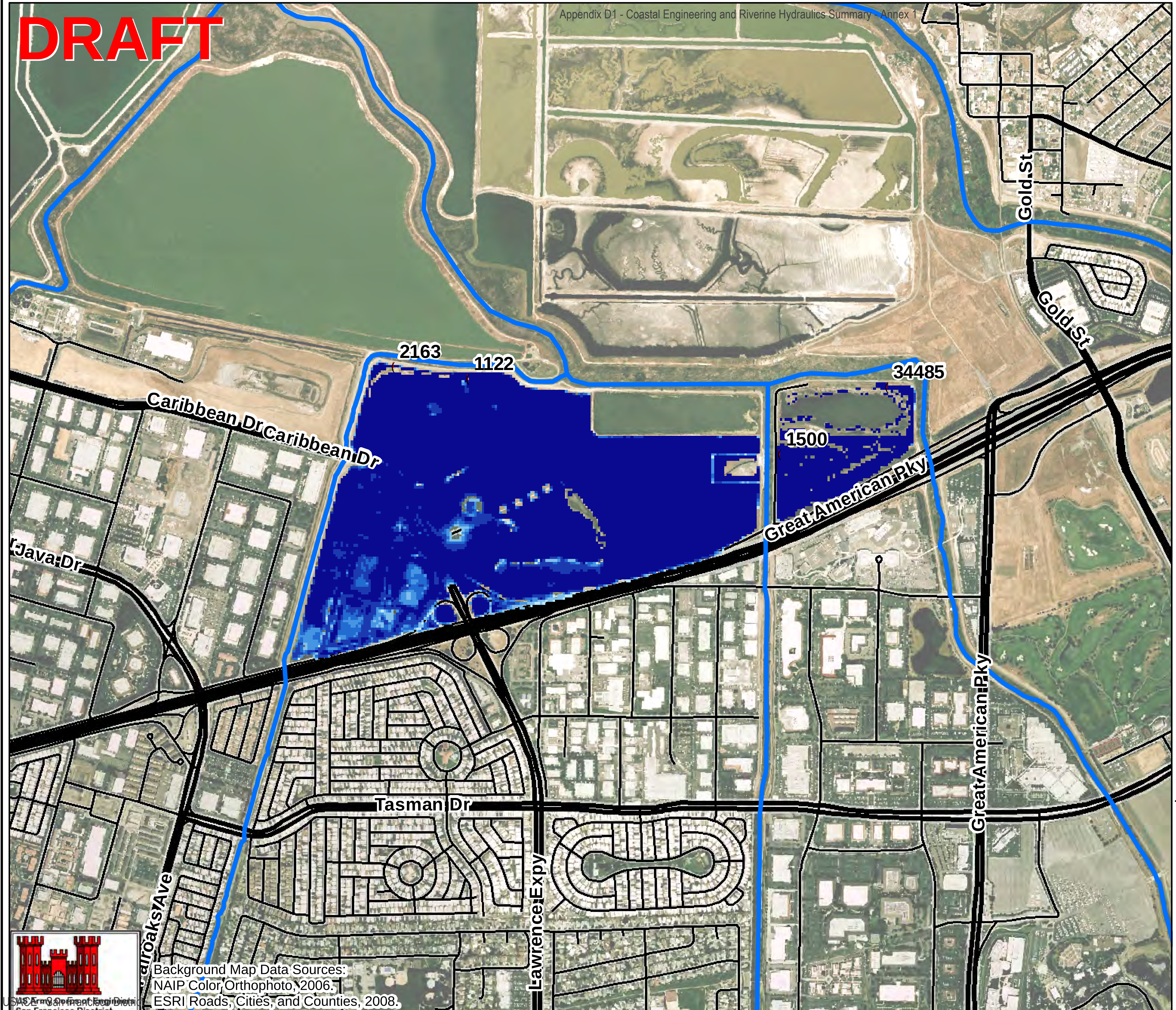
Max Water Depth (ft)

- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 1.5
- 1.5 - 2.0
- 2.0 - 3.0
- 3.0 - 9.0

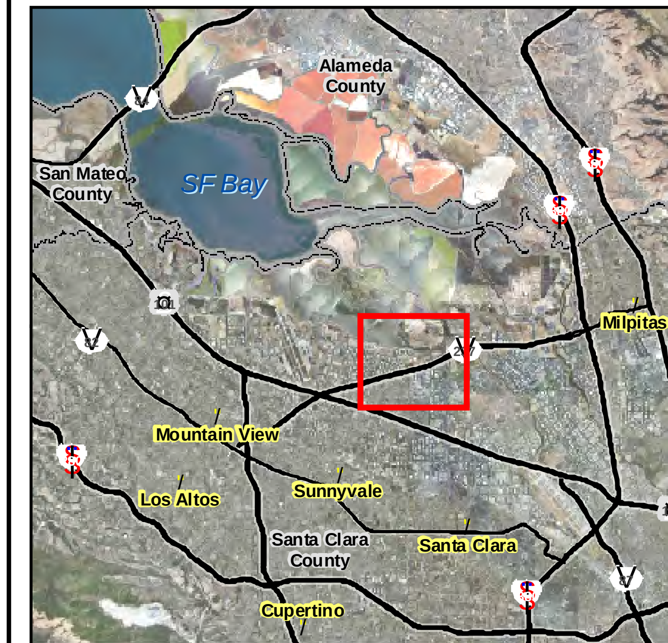
0.4% Exceedance Probability Fluvial Flood Inundation Map

Sunnyvale
Plate 71

US Army Corps of Engineers
San Francisco District



Regional Reference Map



Scale - 1:14,000

Scale - 1 in = 1,166.7 feet

0 875 1,750 3,500 Feet

State Plane CA Zone III	horz. datum: NAD 83 vert. datum: NAVD 88	horz. units: feet 18 Jun 2010

Legend

- Breakout Locations (Stations Labeled)
- Stream Centerline
- Roads

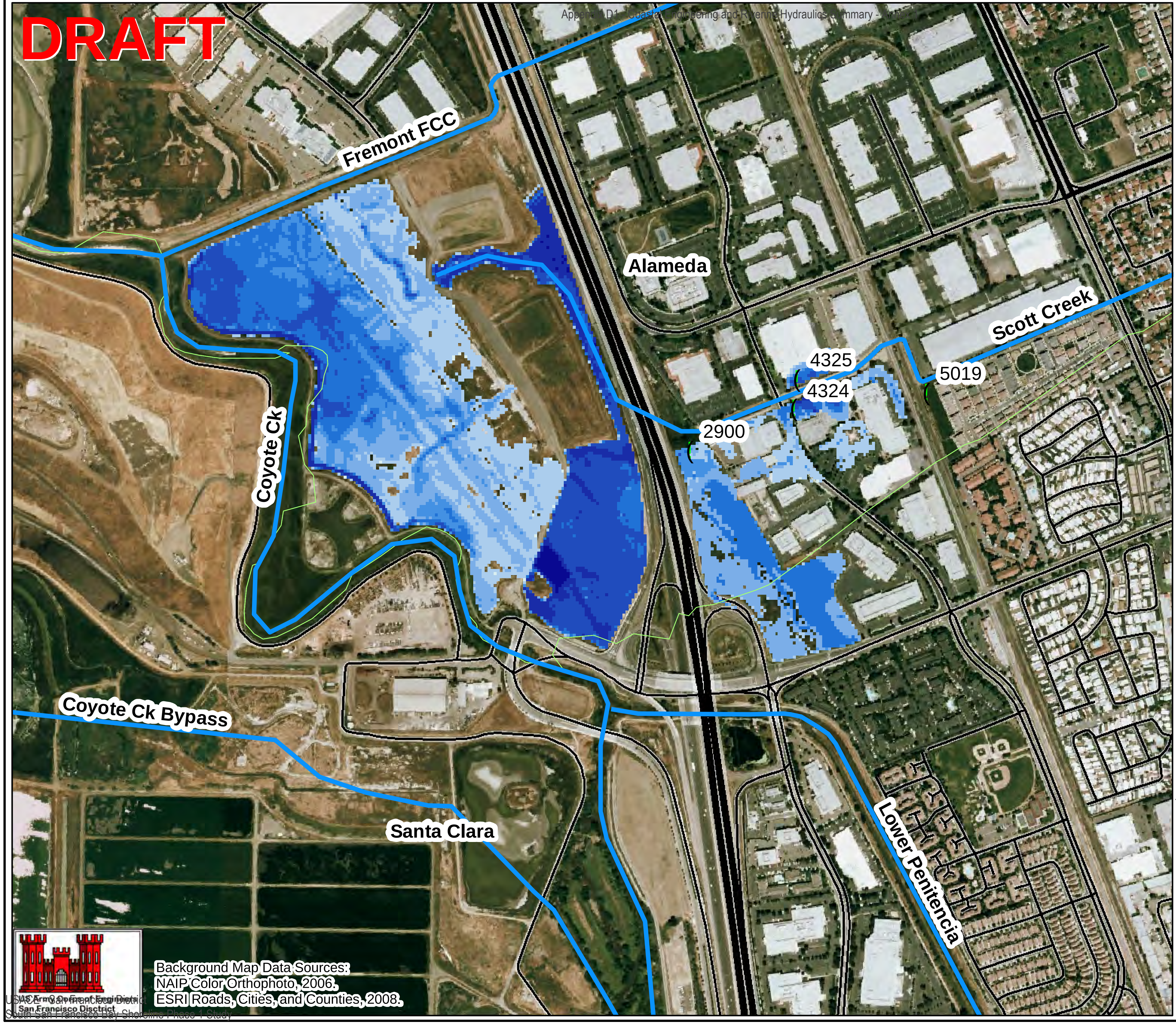
Max Water Depth (ft)

- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 1.5
- 1.5 - 2.0
- 2.0 - 3.0
- 3.0 - 9.0

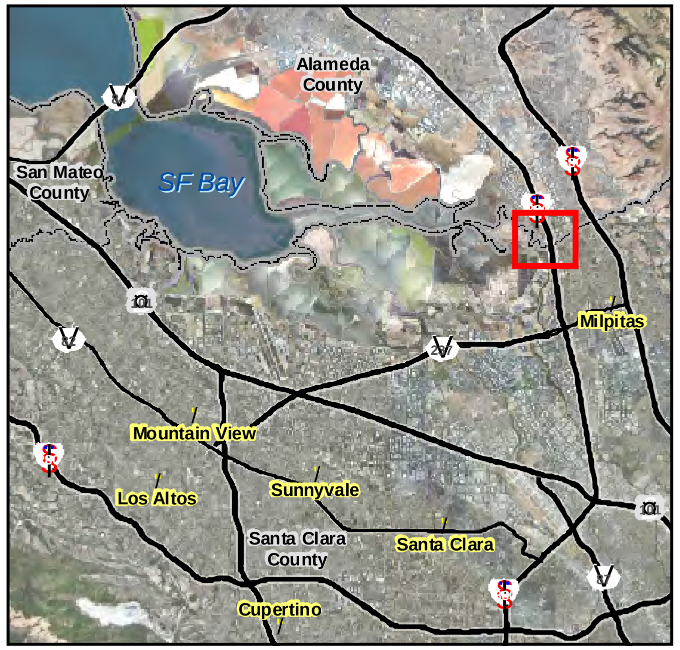
0.2% Exceedance Probability Fluvial Flood Inundation Map

Sunnyvale
Plate 72

US Army Corps of Engineers
San Francisco District



Regional Reference Map



Scale - 1:8,000

Scale - 1 in = 666.7 feet

0 550 1,100 2,200 Feet

State Plane CA Zone III	horz. datum: NAD 83 vert. datum: NAVD 88	horz. units: feet 18 Jun 2010

Legend

- Breakouts
- County Line
- Flood Damage Reductdion Channel

Flow Depth (ft)

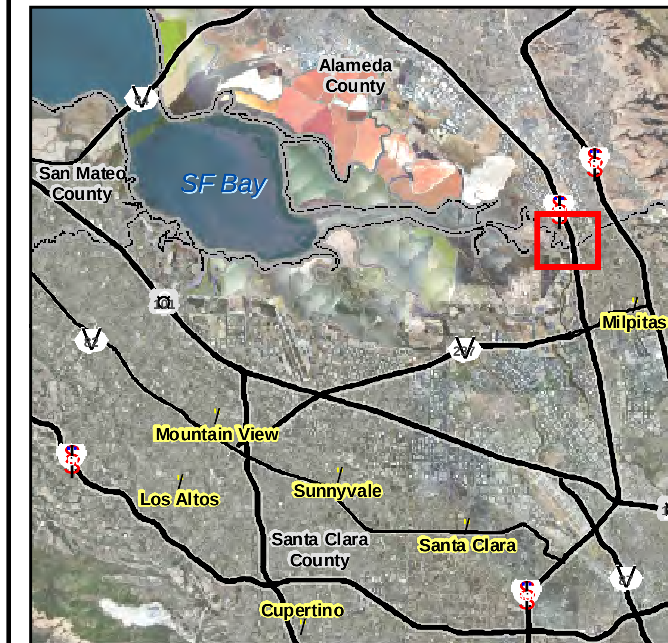
- 0 - 0.5
- 0.6 - 1
- 1.1 - 1.5
- 1.6 - 2
- 2.1 - 3
- 3.1 - 4
- 4.1 - 6
- 6+

1% Exceedance Probability Fluvial Flood Inundation Map

Scott Creek
Plate 73

US Army Corps of Engineers
San Francisco District

Regional Reference Map



Scale - 1:8,000

Scale - 1 in = 666.7 feet

0 550 1,100 2,200 Feet

State Plane CA Zone III	horz. datum: NAD 83 vert. datum: NAVD 88	horz. units: feet 18 Jun 2010

Legend

(Breakouts

County Line

Flood Damage Reductdion Channel

Flow Depth (ft)

- 0 - 0.5
- 0.6 - 1
- 1.1 - 1.5
- 1.6 - 2
- 2.1 - 3
- 3.1 - 4
- 4.1 - 6
- 6+

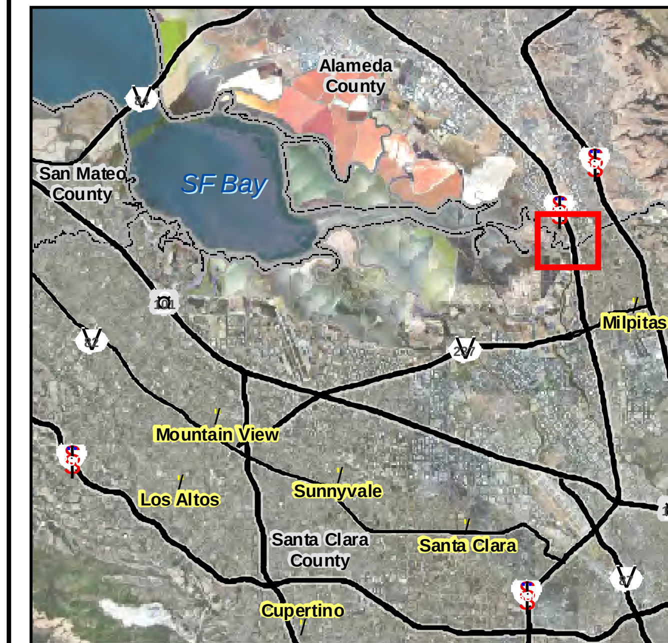
0.4% Exceedance Probability Fluvial Flood Inundation Map

Scott Creek
Plate 74

US Army Corps of Engineers
San Francisco District

US Army Corps of Engineers
San Francisco District

Regional Reference Map



Scale - 1:8,000

Scale - 1 in = 666.7 feet

0 550 1,100 2,200 Feet

State Plane CA Zone III	horz. datum: NAD 83 vert. datum: NAVD 88	horz. units: feet 18 Jun 2010

Legend

- Breakouts
- County Line
- Flood Damage Reductdion Channel

Flow Depth (ft)

- 0 - 0.5
- 0.6 - 1
- 1.1 - 1.5
- 1.6 - 2
- 2.1 - 3
- 3.1 - 4
- 4.1 - 6
- 6+

0.2% Exceedance Probability Fluvial Flood Inundation Map

Scott Creek
Plate 75

US Army Corps of Engineers
San Francisco District