Upper Berryessa Creek Flood Risk Management Project
Milpitas, California

Design Documentation Report

Prepared for:

U.S. Army Corps of Engineers
San Francisco District

In partnership with:

Santa Clara Valley Water District

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100% Submittal

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PROJECT BACKGROUND

This Design Documentation Report (DDR) presents the results of the design of the Upper Berryessa Creek Project along 2.2 miles of Berryessa Creek extending from Interstate 680 (I-680) to Calaveras Boulevard (Highway 237). The design presented follows what is presented in the *Berryessa Creek Element Final General Reevaluation Report/Environmental Impact Statement (GRR/EIS), Santa Clara County, California*, dated March 2014.

This project is being developed under the authority of the Water Resources Development Act of 1990, Public Law (PL) No. 101-640, §101(a)(1), 103 Stat. 4604. The local sponsor is the Santa Clara Valley Water District (SCVWD).

The Berryessa Creek watershed is located in Santa Clara County, California south of San Francisco Bay. Berryessa Creek is a tributary to the Coyote Creek system, which flows into the southernmost end of San Francisco Bay. The Berryessa Creek is a single-purpose flood risk management project and is an element of the Coyote Creek and Berryessa Creeks flood control project authorized by Congress in 1990. The authorized project extends approximately 4.5 miles along Berryessa Creek from 600 feet upstream of Old Piedmont Road to 50 feet downstream of Calaveras Boulevard.

After Congressional authorization in Water Resources Development Act (WRDA) 1990, discussion with the SCVWD and interested environmental groups and community members indicated that the project did not have economic justification or wide support in the community. During preconstruction engineering studies in 1993, project refinements sought to alleviate adverse effects using a rectangular concrete channel to minimize removal of the riparian zone in the upstream reach. Again, this refined project was met with opposition from the community and was subsequently not considered for construction. Furthermore, refined costs and benefits resulted in a project with costs exceeding the benefits, thereby precluding Federal involvement.

In 2001, the SCVWD requested that the U.S. Army Corps of Engineers (Corps) reevaluate flood risk management alternatives along Berryessa Creek to find a more economical and environmentally acceptable solution. The reevaluation renewed public and non-Federal sponsor support for the project. The GRR/EIS was initiated to assess the feasibility of modifying the federally authorized project to reduce flood risks in the Berryessa Creek study area. During the course of the GRR/EIS, Berryessa Creek was separated in two distinct geographic areas: upstream of I-680 and downstream of I-680. The analysis indicated that no flood risk management alternative upstream of I-680 is economically justified. Thus, in 2013, the reach downstream of I-680 was proposed for implementation as a stand-alone element of the authorized project.

The selected plan recommended for implementation would provide capacity to convey a median 0.01 exceedance probability discharge from I-680 to Calaveras Boulevard and would cost approximately $16.6 million. The plan would consist of an earthen trapezoidal channel section with varying bottom widths and 2H: 1V side slopes. Free-standing concrete floodwalls would be constructed, as needed, due to right-of-way (ROW) constraints with in-channel access road constructed where suitable. The existing railroad trestle would be replaced with a double cell reinforced concrete box culvert.

REPORTS PREVIOUSLY ISSUED

Reports previously issued by the Corps are as follows:

a. *"Interim Feasibility Report and Environmental Impact Statement, Coyote Creek and Berryessa Creek, Santa Clara County, California,"* U.S. Army Corps of Engineers, San Francisco District, November 1987.

b. *"Draft General Design Memorandum, Coyote and Berryessa Creeks, Volume 1 of II (Berryessa Creek), California,"* U.S. Army Corps of Engineers, Sacramento District, November 1993.

c. *"Draft General Design Memorandum, Coyote and Berryessa Creeks, Volume II of II, California,"* U.S. Army Corps of Engineers, Sacramento District, November 1993.

d. *"Berryessa Creek Element Coyote and Berryessa Creek California Flood Control Project, Santa Clara County, California. Final General Reevaluation Report and Environmental Impact Statement,"* U.S. Army Corps of Engineers, Sacramento District, March 2014.

e. *"Supplemental Information Report for the Berryessa Creek Element Coyote and Berryessa Creek California Flood Control Project, Santa Clara County, California."* U.S. Army Corps of Engineers and Santa Clara Valley Water District, 30 September 2015.

REFERENCES

1. EM 1110-1-1807, *"Standards Manual for USACE Computer-Aided Design and Drafting (CADD) Systems,"* 30 July 1990.

2. EM 1110-2-1302, "*Civil Works Cost Engineering,"* U.S. Army Corps of Engineers, 31 March 1994.

3. EM 1110-2-1601, *"Hydraulic Design of Flood Control Channels,"* U.S. Army Corps of Engineers, 30 June, 1994.

4. EM 1110-2-2000, *"Standard Practice for concrete for Civil Works Structures,"* U.S. Army Corps of Engineers, Change 2, 31 March 2001.

5. EM 1110-2-2007, *"Structural Design of Concrete Lined Flood Control Channels,"* U.S. Army Corps of Engineers, 30 April 1995.

6. EM 1110-2-2102, *"Waterstops and Other Preformed Joint Materials for Civil Works Structures"*, 30 September 1995.

7. EM 1110-2-2104, *"Strength Design for Reinforced Concrete Hydraulic Structures,"* June 1992.

8. EM 1110-2-2502, *"Retaining and Flood Walls,"* 29 September 1989.

9. ER 1110-2-1150, "*Engineering and Design for Civil Works Projects,"* 31 August 1999.

10. ER 1110-2-1806, *"Earthquake Design and Evaluation for Civil Works Projects,"* 31 July 1995.

11. ETL 1110-2-256, "*Sliding Stability for Concrete Structures,"* 24 June 1981.

12. ETL 1110-2-322, *"Retaining and Flood Walls,"* October 1990.

13. ETL 1110-2-307, *"Flotation Stability for Concrete Hydraulic Structures,"* 20 August 1987.

14. Design Guide, *"Structural Design of Flood Control Channels," U.S. Army Corps of Engineers, Los Angeles Districts, October 29, 1998.*

15. American Concrete Institute, *"Building Code Requirements for Reinforced Concrete, ACI 318M-95 and Commentary, ACI 318RM-95."*

16. American Institute of Steel Construction (AISC), *"Manual of Steel Construction,"* ASD, Ninth Edition, 1989.

17. Tetra Tech. March 2015. Wetlands/Other Waters of the U.S./Waters of the State Delineation Report, Prepared for Santa Clara Valley Water District.

18. USACE. 2014. Final General Reevaluation Report and Environmental Impact Statement, Berryessa Creek Element, Coyote and Berryessa Creek, California, Flood Control Project

19. Waterways Experimental Station (WES), Corps of Engineers Computer Program, *"Concrete Strength Investigation and Design (CASTR),"* May 1987.

20. Waterways Experimental Station (WES), Corps of Engineers Computer Program, *"Analysis of Retaining and Flood Walls (CTWALL),"* 30 October 1993.

PERTINENT DATA

Purpose: Flood Risk Management

Item	Description
Berryessa Creek Drainage Area (Entire Reach Length: 4.5 miles)	22.4 square miles
Project Creek Length	2.2 miles
100-Year Peak Discharge	4,100 cfs
Design Discharge	4,100 cfs
Reach Lengths	
I-680 to Montague Expressway (Stream Miles 3.81 to 3.15)	3,450 feet
Montague Expressway to Piedmont Creek (Stream Miles 3.15 to 2.18)	5,150 feet
Piedmont Creek to Los Coches Street Bridge (Stream Miles 2.18 to 1.77)	2,150 feet
Los Coches Street Bridge to Approximately 50 feet downstream of Calaveras Boulevard (Stream Miles 1.77 to 1.68)	500 feet
50 feet downstream of Calaveras Boulevard to the Confluence with Lower Penitencia Creek (Stream Miles 1.68 to 0.00)	8,850 feet

ABBREVIATIONS AND ACRONYMS

AST	above-ground storage tank
AW	Ackers-White
BMP	best management practice
CalOSHA	California Division of Occupational Safety and Health
RWQCB	California Regional Water Quality Control Board
cfs	cubic feet per second
CERCLA	Comprehensive Environmental Response, Compensation, and Liability Act
CIE	channel improvement easement
CPT	cone penetration test
C&M	Construction and Maintenance
CMP	corrugated metal pipe
CNP	conditional non-exceedance probability
DDR	Design Documentation Report
EIR	Environmental Impact Report
EIS	environmental impact statement
EM	Engineer Memorandum
ER	Engineer Regulation
ESL	ecological screening level
ETL	Engineering Technical Letter
FEMA	Federal Emergency Management Agency
FPLE	flood protection levee easement
GDM	General Design Memorandum
GIS	Geographical Information Systems
GRR	general reevaluation report
LiDAR	Light Detection and Ranging
MCE	Maximum Considered Earthquake
MDE	Maximum Design Earthquake
MPM	Meyer-Peter-Muller
NAD	North American Datum
NAT	North American Transformer
NAVD	North American Vertical Datum
NED	National Economic Development
NEPA	National Environmental Policy Act
NFIP	National Flood Insurance Program
NGA	Next Generation Attenuation
OBE	Operating Basis Earthquake
O&M	operation and maintenance

OMRR&R	operation, maintenance, repair, rehabilitation, and replacement
PED	Preconstruction Engineering and Design
PL	Public Law
RCBC	reinforced concrete box culvert
RCP	reinforced concrete pipe
ROW	right-of-way
RWQCB	Regional Water Quality Control Board
§	subsection
SCVWD	Santa Clara Valley Water District
SPT	standard penetration test
Stat.	Statute
TCA	trichloroethane
TPH-d	total petroleum hydrocarbons as diesel
TPH-g	total petroleum hydrocarbons as gasoline
TWAE	temporary work area easement
UPRR	Union Pacific Railroad
USACE	United States Army Corps of Engineers (Corps)
USC	United States Code
USEPA	United States Environmental Protection Agency
USFWS	United States Fish and Wildlife Service
USGS	United States Geological Survey
UST	underground storage tank
VE	value engineering
VOC	volatile organic compound
VTA	Valley Transportation Authority
WRDA	Water Resources Development Act

1. INTRODUCTION

GENERAL

1.1 The Berryessa Creek drainage basin covers 22.4 square miles in northeastern Santa Clara County (Figure 1.1). Berryessa Creek flows westerly from its origin in Mt. Hamilton of the Diablo Range through the cities of San Jose and Milpitas. It then turns north and discharges into Lower Penitencia Creek, which is a tributary to Coyote Creek that flows into San Francisco Bay.

1.2 The Berryessa Creek watershed is located in Santa Clara County, California, south of San Francisco Bay (Figure 1.2). As shown in Figure 1.3, the project, as authorized in 1990, begins 500 feet upstream of the upstream face of Old Piedmont Road to 50 feet downstream of Calaveras Boulevard Bridge.

1.3 As part of the general reevaluation study completed in 2014, the authorized project reach was separated in two distinct geographic areas: upstream of I-680 and downstream of I-680. The analysis indicated that no flood risk management alternative upstream of I-680 is economically justified. Thus, the reach downstream of I-680 was proposed for implementation as a stand-alone element of the authorized project. The proposed downstream improvements are noted in Figure 4.1.

PROJECT AUTHORIZATION

1.4 In 1989, the Chief of Engineers transmitted an Interim Feasibility Report for Coyote Creek and Berryessa Creek to Congress. The Berryessa Creek Element was authorized by the WRDA of 1990, PL No. 101-640, §101(a)(5), 103 Stat. 4604 (1990), which states:

> *"(a) Projects With Report of the Chief of Engineers. – Except as provided in this subsection, the following projects for water resources development and conservation and other purposes are authorized to be carried out by the Secretary substantially in accordance with the plans, and subject to the conditions, recommended in the respective reports designated in this subsection:*
>
> > *(5) Coyote and Berryessa Creeks, California. – The project for flood control, Coyote and Berryessa Creeks, California: Report of the Chief of Engineers, dated February 7, 1989, at a total cost of $56,300,000, with an estimated first Federal cost of $39,000,000 and estimated first non-Federal cost of $17,300,000."*

1.5 After Congressional authorization in 1990, discussions with the SCVWD and interested environmental groups and community members showed that the project did not have wide support in the community. Preconstruction engineering and design efforts resulted in project refinements that had higher costs than benefits, and work stopped in 1993. In 2001, the SCVWD requested that the Corps reevaluate existing flooding potential along Berryessa Creek to reassess potential federal interest to implement the authorized flood risk management measures.

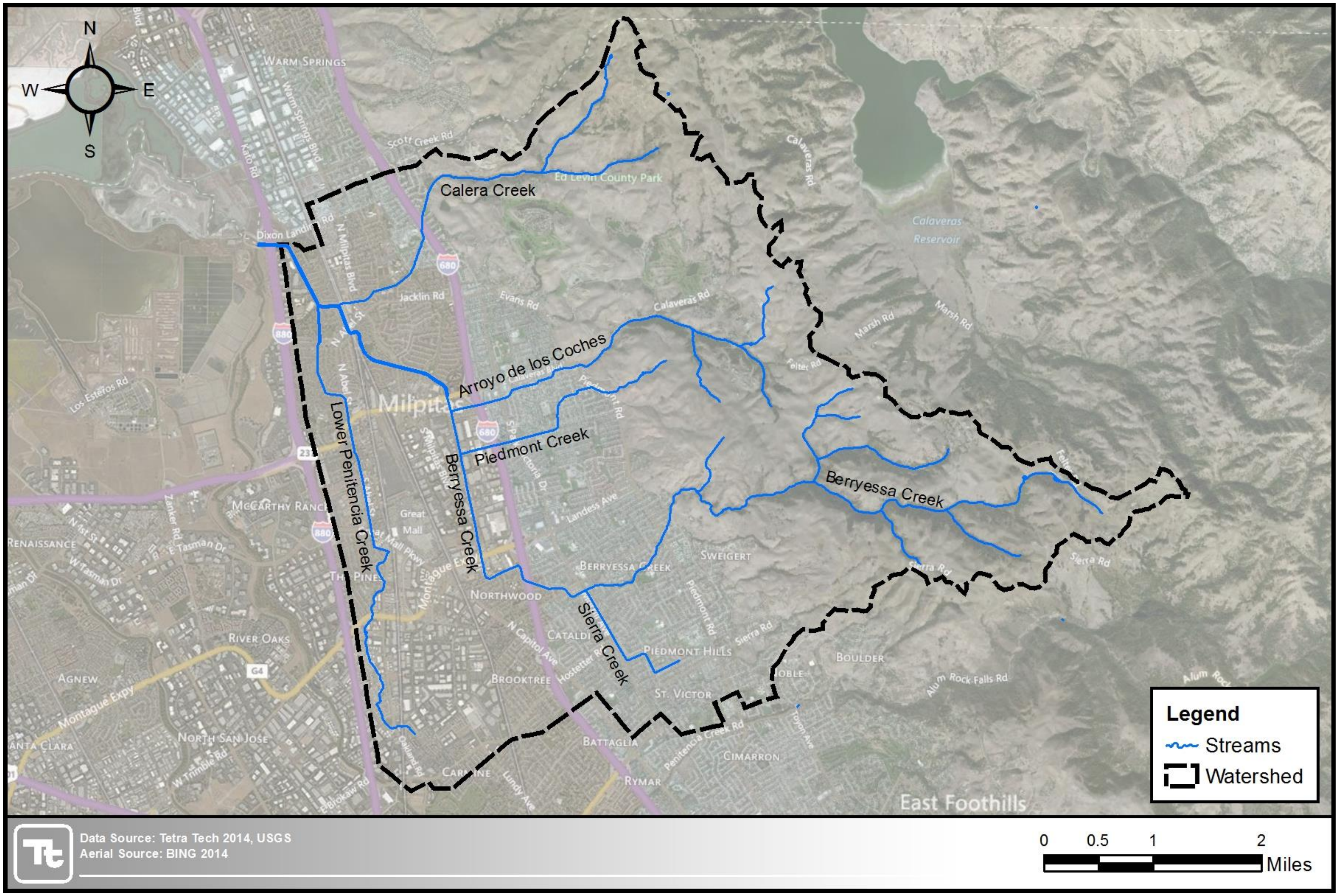

Figure 1.1 Berryessa Creek Watershed

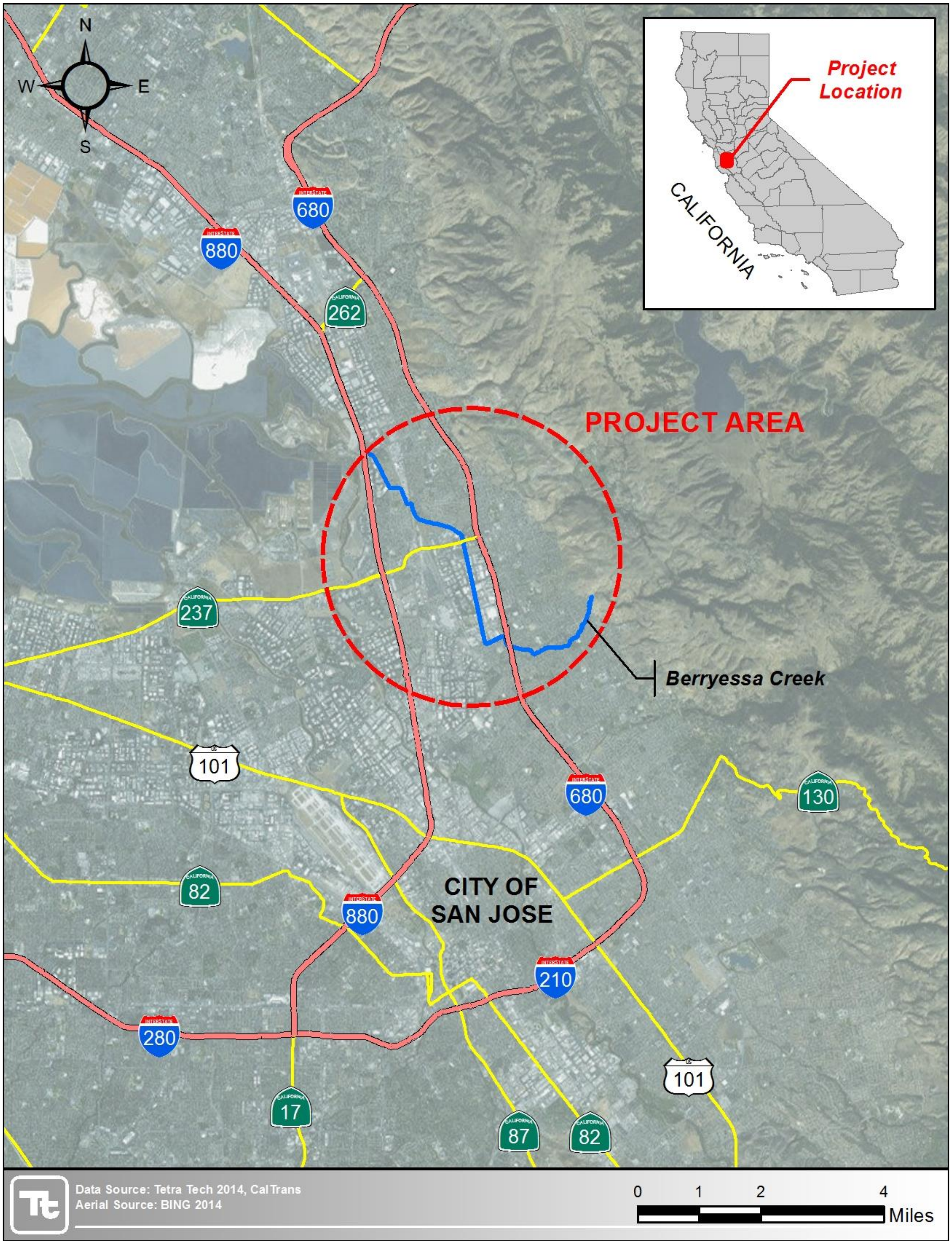

Figure 1.2 Location Map

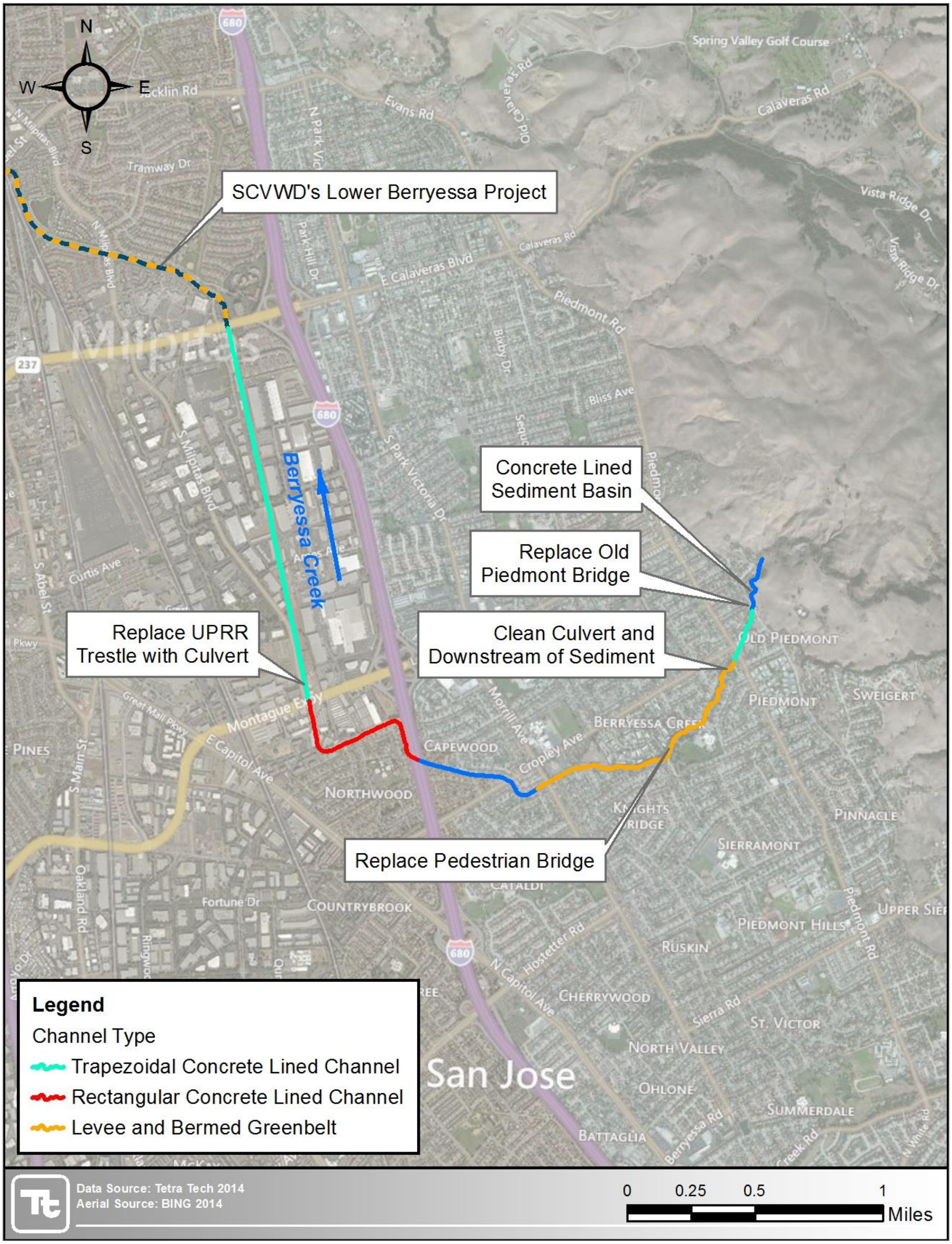

Figure 1.3 Original Authorized Project

PURPOSE

1.6 The purpose of this DDR is to provide the basis for design of the flood risk management project along approximately 2.2 miles of Berryessa Creek that extends from I-680 to Calaveras Boulevard. The project would provide capacity to convey the median 0.01 exceedance probability discharge and would consist of an earthen trapezoidal channel section with varying bottom widths and 2H: 1V side slopes.

SCOPE OF STUDIES

General

1.7 This DDR presents the design for the recommended plan, the estimated construction cost, and the schedule for the Upper Berryessa Creek flood risk management project. The Berryessa Creek drainage basin covers 22.4 square miles in northeastern Santa Clara County. Berryessa Creek flows westerly from its origin in Mt. Hamilton of the Diablo Range through the cities of San Jose and Milpitas. It then turns north and discharges into Lower Penitencia Creek, which is a tributary to Coyote Creek that flows into San Francisco Bay. The watershed area in the Diablo Range is comprised of clay surface soils that are potentially highly erodible and are subject to slope failure, settlement, and sedimentation. The basin consists of a large proportion of flat valley and foothill areas that have been rapidly urbanized. It also consists of a significant percentage of steep mountainous areas that are utilized primarily for agriculture and resource extraction.

1.8 The study watershed is divided by I-680, located approximately midway within the watershed, separating Berryessa Creek into two distinct study sub-watersheds. In the vicinity of the study area, I-680 is constructed on a raised embankment with concrete sound walls along each side of the freeway right-of-way. Thus, the as-constructed freeway creates an impediment to overland flooding from continuing to the lower portions of the watershed. The only conveyance pathway is the existing Berryessa Creek culvert under the freeway.

1.9 The project addressed in this DDR is the 2.2 miles of Berryessa Creek extending downstream of I-680 to Calaveras Boulevard.

Surveying and Mapping

1.10 The mapping is based on Light Detection and Ranging (LiDAR) method aerial topography developed by Photo Science Inc., in 2013 for SCVWD. The LiDAR data was provided as point data in XYZ format text files with associated one-foot contours by SCVWD. Horizontal control is based on the North American Datum (NAD) of 1983, California State Plane 3; vertical control is based on the North American Vertical Datum (NAVD) of 1988.

Site Explorations

1.11 Subsurface investigations have been performed by Tetra Tech, under contract to the SCVWD for the design of the Berryessa Creek Project. Analyses and results are presented in the Geotechnical Appendix.

Coordination with Others

1.12 Coordination of the design of the project will be conducted with appropriate agencies, partners, stakeholders and utilities as noted in this subsection. Items that would be discussed include mapping, as-built plans, ROW, historic properties, easements, utility relocations, water quality and nonpoint source discharges, cleanup of contaminated areas, tree replanting plan, and potential sources of water, disposal sites, and maintenance features.

a. Ongoing coordination with the local sponsor, SCVWD.

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• Rights-of-Way. The real estate interests required for and/or impacted by the project are owned and/or held by private owners, county governments or municipalities, public and private utilities, and the Union Pacific Railroad (UPRR). ROW requirements are established in detail and included in the Real Estate section of this report.

• Utility Relocations. The project would require relocations of publicly and privately owned utilities. Interfering utilities include electrical, water lines, telephone, sanitary sewer, and cable. Numerous utilities are located throughout the project including along bridge crossings and adjacent to the channel improvements. Unless otherwise identity all existing utilities will be protected in place. Where possible, relocations will be accomplished in advance of the construction. The affected utilities requiring relocation are identified in Table 1-1.

Table 1-1 Utility Relocations

Berryessa Creek Reach	Location	Type	Owner
Montague to UPRR Trestle – Channel	Sta. 162+00 to 164+00	Monitoring Wells	
UPRR Trestle to Culvert – Channel	Sta. 152+50	Electrical Vault/Lines	PG&E
Ames to Yosemite – Channel	Sta. 133+17	Sanitary Sewer System Manhole Adjustment	City of Milpitas
	Sta. 131+67	Monitoring Well	
	Sta. 129+12	Sanitary Sewer System Manhole Adjustment	City of Milpitas
	Sta. 125+15	Sanitary Sewer System Manhole Adjustment	City of Milpitas
Yosemite to Los Coches – Channel	Sta. 120+65	Sanitary Sewer System Manhole Adjustment	City of Milpitas
	Sta. 118+85	Monitoring Well	
	Sta. 116+25	Sanitary Sewer System Manhole Adjustment	City of Milpitas
	Sta. 114+85	Sanitary Sewer System Manhole Adjustment	City of Milpitas
	Sta. 111+75	Sanitary Sewer System Manhole Adjustment	City of Milpitas

Table 1-1 Utility Relocations

Berryessa Creek Reach	Location	Type	Owner
	Sta. 109+20	Sanitary Sewer System Manhole Adjustment	City of Milpitas
	Sta. 107+25	Sanitary Sewer System Manhole Adjustment	City of Milpitas
	Sta. 102+80	Sanitary Sewer System Manhole Adjustment	City of Milpitas
	Sta. 97+85	Sanitary Sewer System Manhole Adjustment	City of Milpitas
	Sta. 97+80	Irrigation Box	To be abandoned
Los Coches to Calaveras - Channel	Sta. 93+75	Sanitary Sewer System Manhole Adjustment	City of Milpitas
	Sta. 92+65	Sanitary Sewer System Manhole Adjustment	City of Milpitas
	Sta. 89+65	Sanitary Sewer System Manhole Adjustment	City of Milpitas
	Sta. 88+89	Gauging Vault	

- The project proposes to modify local storm drain outlets to allow for a controlled outlet into the channel. The outlet structures will maintain the existing storm drain pipe size and connect to the channel. Storm drain sizes vary from 8” corrugated metal pipe (CMP) to 36” reinforced concrete pipe (RCP). For the project, the proposed connections will utilize a minimum 18-inch RCP for maintenance purposes. Affected storm drain outlets are identified in Table 1-12.

Table 1-2 Storm Drain Outlets

	EXISTING PIPE		NEW PIPE	
STATION	TYPE	DIAMETER (in.)	TYPE	DIAMETER (in.)
87+64.30	CMP	24	RCP	24
90+16.85	CMP	18	RCP	18
92+63.07	CMP	12	RCP	18
97+64.58	RCP	36	RCP	36
98+88.87	CMP	24	RCP	24
109+27.35	CMP	24	RCP	24
137+45.87	CMP	30	RCP	30
137+76.54	CMP	8	RCP	18
151+03.88	CMP	15	RCP	18
151+78.24	RCP	30	RCP	30
155+94.49	CMP	10	RCP	18
162+42.18	CMP	18	RCP	18
163+44.63	CMP	10	RCP	18
163+77.93	CMP	18	RCP	18
171+38.56	CMP	18	RCP	18
172+95.03	CMP	18	RCP	18
174+66.42	CMP	18	RCP	18
178+69.78	CMP	15	RCP	18
183+04.17	CMP	18	RCP	18
186+55.09	CMP	18	RCP	18
190+63.17	CMP	18	RCP	18

- Sewer crossings located at Los Coches and Piedmont Creek, will be partially modified to allow for the construction of a concrete channel over the existing lines. The proposed modification consist of a partial removal of the top of the existing concrete encasement (approximately 4 inches). The bottom of the invert slab of the proposed concrete channel will be notched and a 2 inches of styrofoam spacer provided to prevent a transfer of loading to the sewer line from the proposed improvements. The proposed concrete channels will provide additional protection to the existing exposed VCP and remaining concrete encasement. Additionally, the sewer manholes located along the right bank from Ames to Calaveras will be adjusted to meet the proposed access road finish grade.

- The existing City of Milpitas waterlines are to be protected in place and the finished grades of the channel will allow the pipelines to remain without modification. The City is proposing to install two new waterline crossings at Los Coches Street. These improvements will be coordinated with the project so that the portions crossing the sewer could be installed prior to the construction of the concrete transition structures.

- The existing City of Milpitas exercise pocket park (Station 94+00) will be removed to allow for the construction of the channel improvements and 18-foot-wide access road.

- Other modifications consisting of grading along the existing UPRR spur line will be performed to provide positive drainage away from the track towards the creek and avoid the need for inlets and drainage swales.

- For dry utilities (cable, electrical, telephone, fiber optic, and gas) secondary notices where sent off requesting confirmation of facilities and existing vertical design. Coordination with the utility agencies and potholing was performed on 3 PG&E facilities crossing the channel to determine the location of the facilities and relocation requirements. Based upon the results only the PG&E facility located at approximate Station 152+50 will require relocation. In addition to this facility two utility vaults located within Los Coches Street will be relocated to construct a driveway for the access roads. Table 1-1 identifies the three dry utilities that require relocation or modification. The required relocations will be performed by the utility company. Based upon whom has prior rights, the cost of utility relocation will be determined and updated in the construction cost estimate.

- Some properties required for and/or impacted by the project are located within, adjacent to, or close to SCVWD's existing ROW along the Berryessa Creek. A strip of right of way downstream of Montague Expressway, between the channel and Milpitas Blvd., is required to complete the channel and access road improvements. Additionally, maintenance easements below Los Coches Street, Yosemite Drive, and Ames Avenue are required from the City of Milpitas. Temporary grading along the channel improvements adjacent to UPRR property will be obtained through the permitting process with UPRR. A detailed description of ROW requirements and acreages is provided in the Real Estate Plan in Section 13.

- Maintenance Items. Required maintenance features will be coordinated with the local sponsor and the project team.

Maintenance Access. Per coordination with the local sponsor, the preferable width for access roads is 18 feet wide and constructed of an appropriate aggregate base material. Due to ROW limitations, the access road widths are reduced to 15 feet or less, and in some locations, are only provided on one side of the channel. Due to the limited ability, only existing at-grade access road crossings of the railroad tracks will remain. Fencing and gates will be provided to prevent unauthorized crossing of the rail road lines.

b. Coordination with other agencies included:

- U.S. Fish and Wildlife Service
- California Department of Fish and Wildlife
- San Francisco Bay Regional California Water Quality Control Board (RWQCB)
- California Department of Transportation
- State Office of Historic Preservation
- Native American Heritage Commission
- City of Milpitas
- City of San Jose
- Santa Clara County
- Bay Area Rapid Transit
- Union Pacific Railroad
- Valley Transportation Authority

2. DESCRIPTION OF DESIGN IN GENERAL REEVALUATION REPORT

GENERAL

2.1 The analysis completed for the GRR/EIS in 2014 indicated that no flood risk management alternative upstream of I-680 was economically justified. Thus, the reach downstream of I-680 was proposed for implementation as a separable element of the authorized project. The tentatively selected plan recommended for implementation was the NED Plan identified as Alternative 2A/d.

2.2 The tentatively selected plan, as described in the GRR/EIS, consists of a 0.01 exceedance probability event level of performance, with 76 percent assurance, downstream of the I-680 culvert. Alternatively, based on interpolation, at an assurance level of 90 percent, the plan would be able to contain the equivalent of about a 0.03 exceedance probability event.

2.3 The plan would consist of an earthen trapezoidal channel section with varying bottom widths and 2H: 1V side slopes. Due to real estate constraints, free-standing concrete floodwalls would be constructed instead of levees in the immediate vicinity of Montague Expressway, as well as between the Piedmont Creek confluence and Calaveras Boulevard. Concrete floodwalls would include 42-inch safety railing for any wall heights above two feet. An access road would be located along the left bank of the channel downstream of Yosemite Avenue. Transition structures at Montague Expressway, UPRR culvert, Los Coches Street, and Calaveras Boulevard would be constructed. Transition structures (with variable sloping wingwalls) would extend for 50 to 75 feet upstream or downstream of the bridge face. The existing UPRR trestle would be replaced with a triple barrel reinforced concrete box culvert. Storm drains entering the channel, or running parallel to the channel, and situated within the proposed channel excavation areas would be relocated.

CHANNEL MODIFICATIONS

2.4 Channel widening is proposed in combination with floodwalls to meet the desired level of conveyance performance for the project. The extent of armoring, including toe down depths and armor rock gradation, may vary from section to section as the design is refined. In narrow reaches, the toe protection may be continuous across the channel bottom to maintain the integrity of the channel. To prevent downcutting of the channel bed, the channel profile may require construction of grade control elements at bridge or utility crossing locations.

2.5 The project includes an intermittent access road within the channel at the approximate level of 0.1 to 0.04 exceedance probability event. The access road would increase the effective conveyance area within the available ROW for larger events and allow maintenance equipment to have closer access to the channel. The access road surface would need to be paved or graded and compacted to withstand flood flows, and the road surface would require a cross slope for drainage. The access road location is generally proposed on the left bank in the cross sections but alternatively may be located on the right bank if deemed appropriate; a secondary access road may be located along the opposite bank.

BRIDGES

2.6 The project proposes the construction of transitions from the proposed flood walls to the existing wingwalls at Montague Expressway, UPRR culvert, Los Coches Street (including the pedestrian bridge), and Calaveras Boulevard. These wingwalls would provide transitions between the proposed channel/floodwalls and the existing bridge structures and provide for continued structural integrity of the bridge foundations and abutments. Additionally, abutment and pier protection is planned for the bridges at Ames Avenue and Yosemite Drive. The design would protects the piers/abutments from increased flow volumes and velocities creating potential undermining that could result from the planned deepening of the channel at these locations.

2.7 The project proposes the replacement of the existing railroad trestle (Station 161+00) with a triple box culvert. The concrete culvert would have openings at approximately 10 feet by 11 feet and will be cast-in-place with steel reinforcement. New railroad tracks will need to be rebuilt on top of the new triple barrel, reinforced concrete box culvert. New ballast rock will be brought in along with new primary rails and wooden ties.

3. VALUE ENGINEERING STUDY

3.1 During the completion of the 30 percent design, the Value Engineering (VE) Team (including Value Management Strategies, Inc., Noble Consultants, Inc., and representatives from the Corps and SCVWD) conducted a VE study for the U.S. Army Corps of Engineers – San Francisco District on the Berryessa Creek Flood Control Project. The study was conducted in the San Francisco area in October 2014. The overview of the project, key findings of the study, and an overview of the alternatives developed by the VE team are summarized in the report provided in **Appendix G**.

3.2 The VE study identified 11 alternatives that could potentially add value to the project, either through cost savings, reducing the project delivery schedule, performance improvements, or a combination thereof. These alternatives, as shown in Table 3.1, are organized into the following categories based on the project issue or project aspect being addressed by them.

- Two earthwork quantity and material handling/disposal alternatives
- Three alternative methods for channel erosion control
- Two alternatives related to incorporating proposed Montague Expressway Improvements
- Two UPRR trestle bridge replacement alternatives
- Two miscellaneous alternatives

Table 3-1 Alternative No. and Description

Earthwork Alternatives
E-1 Reduce off-site disposal of excavated material and steepen side slopes to balance earthwork cut and fill
E-2 Steepen the side slopes of channel excavation to eliminate floodwalls
Erosion Control Alternatives
EC-1 Reduce height of cellular slope protection to 2- to 5-year event levels
EC-2 Extend cellular mat protection to toe of slope in lieu of riprap
EC-3 Reduce temporary erosion control assumptions for the project
Montague Expressway Improvements Alternatives
M-1 Eliminate the floodwalls upstream of Montague Expressway
M-2 Clean out existing channel upstream of Montague Expressway in lieu of modifying channel
UPRR Trestle Bridge Replacement Alternatives
RR-1 Realign UPRR in permanent location in lieu of temporary shoo-fly
RR-2 Replace UPRR trestle bridge on same alignment without shoo-fly
Miscellaneous Alternatives
Misc-1 Use localized pumping of construction in lieu of dewatering
Misc-2 Use decomposed granite in lieu of AC paving for recreational trail

3.3 <u>E-1 Reduce off-site disposal of excavated material and steepen side slopes to balance earthwork cut and fill</u>: Eliminate or substantially reduce off-haul by allowing for more on-site and out-of-channel material fill by increasing the elevation of adjacent access ways and construction-free zones. Make the channel side slopes steeper and make channel base width wider to preserve or increase channel cross-sectional area while also slightly reducing placement of fill within wetted perimeter of the channel section. After review of this proposal, it was determined that steepening the side slope is not recommended, as stability of channel would become an issue. Based on the geotechnical recommendations, side slopes steeper than 2:1 should be avoided except for short localized areas where required. The project team has looked into finding areas to dispose of material onsite to avoid off-site trucking.

3.4 E-2 Steepen the side slopes of channel excavation to eliminate floodwalls: Steepen the side slopes of channel excavation from 2H: 1V to 1H: 1V to eliminate floodwalls wherever feasible. As mentioned above this recommendation was not incorporated into the design.

3.5 EC-1 Reduce height of cellular slope protection to 2- to 5-year event levels: The alternative would limit the height of the cellular slope protection to approximately the 2- or 5-year water surface elevation. The alternative acknowledges that the majority of erosion potential is from the more frequent events, rather than the design discharge. After review of this proposal, it was determined that this proposal would reduce protection and provide a minimal cost savings for reduction in protection. It should be noted that the cellular slope protection would start on the top of the rip rap section and proceed to the top of the channel. The cellular material will also help against riling of slopes post construction and provide protection during the establishment period of vegetative growth in the channel.

3.6 EC-2 Extend cellular mat protection to toe of slope in lieu of rip rap: This VE alternative is to remove all rip rap and associated geotextile fabric from the channel toe areas and extend the cellular bank protection through the former rip rap area to 5 feet below the channel invert. After review of this proposal, it was determined that this proposal would limit and reduce protection. Additionally, cellular material tends to roll up once a section has failed, which has occurred in the existing project reach at the Piedmont Creek confluence. As part of the 60 percent design a detailed review of the channel, shear stresses was performed, and the height of the rip rap (on the channel embankment) was refined. As part of the GRR, the anticipated protection was proposed at a 4 foot height, based upon this additional analysis, the rip rap height was adjust to a lower elevation based upon the shear stresses near or around 1 lbs/ft^2. This value was considered an acceptable value for the use of cellular protection.

3.7 EC-3 Reduce temporary erosion control assumptions for the project: Provide silt fences and straw rolls in very limited locations (i.e., construction staging areas). After review of this proposal, it was determined that this proposal would be incorporated into the project specifications. The construction contractor is to prepare and provide temporary erosion control to meet the water quality construction permitting requirements. Project specifications will require a SWPPP to be prepared by the construction contractor. This cost will be refined as the project design continues and the limits of work are refined.

3.8 M-1 Eliminate the floodwalls upstream of Montague Expressway: This proposal has been incorporated into the design. With the bridge improvements to Montague Expressway, the hydraulic capacity has been increased and lowered the water surface upstream of the bridge.

3.9 M-2 Clean out existing channel upstream of Montague Expressway in lieu of modifying channel: This proposal has been incorporated into the design.

3.10 RR-1 Realign UPRR in permanent location in lieu of temporary shoo-fly: Construct a realigned new concrete triple barrel, reinforced concrete box culvert at the UPRR crossing immediately adjacent to the existing timber trestle on either the downstream or upstream side, and then remove the existing timber trestle in lieu of constructing a temporary shoo-fly trestle and then removing the temporary trestle. After review of this proposal, it was determined that this proposal would require track realignment and would require property acquisition on the west side of Milpitas Road. In meeting with UPRR, the option of weekend construction to replace the trestle bridge with a pre-cast structure was discussed and would be allowed if the realignment of a shoo-fly was not practical or required property acquisition. Currently, the design will proceed with a bridge replacement along the existing railroad alignment without a shoo-fly track. The project team will pursue coordinating with UPRR on weekend construction, which would eliminate the construction of a shoo-fly track and limit the amount of track impacts, providing cost savings. Additionally, the box culvert size has been reduced from a triple barrel box to a double barrel box, providing construction cost savings.

3.11 RR-2 Replace UPRR trestle bridge on same alignment without shoo-fly: Construct a new steel trestle in place with the same alignment as existing timber trestle. See above.

3.12 Misc-1 Use localized pumping during construction in lieu of dewatering: The alternative would be to provide only limited and localized pumping, if at all. This proposal has been considered and will require further discussion. The proposal is dependent on the amount of water that would need to be treated and/or removed. It should be noted that water depth will be checked to determine seasonal levels and expected water depths during construction. It is anticipated that the water table depths would decrease during the dry periods of the year. Further, since volatile organic compound (VOC) contamination of groundwater may be encountered during construction, specifications will include an option to use a localized treatment facility (carbon/sand filtration) prior to reintroduction as surface water; contractor's cost estimate will also have a line item for a unit cost to treat "X" amount of water that can be included as a contingency item.

3.13 Misc-2 Use decomposed granite in lieu of AC paving for recreational trail: Eliminate the roadway base and AC surfacing of the recreational trail and use alternative material such as graded aggregate base or decomposed granite instead. This proposal has been incorporated into the design. Additionally, at the request of SCVWD, the preferred access road requirement is to utilize aggregate base instead of AC paving since the maintenance equipment can damage AC access roads.

3.14 Of the identified 11 alternatives proposed, only RR-2 is incorporated into the final design.

4. SELECTED PLAN

GENERAL

4.1 The selected plan by USACE for the Berryessa Creek Project (based on the analysis contained in the GRR/EIS), consists of flood risk management improvements for approximately 2.2 miles of Berryessa Creek extending from the I-680 to Calaveras Boulevard (Figure 4.1). The Selected Plan consists of an earthen trapezoidal channel section with varying bottom widths. Free-standing concrete floodwalls would be constructed, as needed, due to ROW constraints, and an in-channel access road will be constructed where suitable. Through the design process, Tetra Tech reviewed the preliminary design as part of the GRR and performed a refinement of the preliminary design. The major plan modifications include streamlining the transitions at the bridge crossings, including concrete lining of the invert and channel side slopes, improvement of the channel junctions with Los Coches Creek and Piedmont Creek, and revisions to the bottom widths to fit the channel into the existing ROW. The revisions and associated hydraulic improvements allowed for the removal of the in-channel access road and a reduction in the length and height of the required floodwall as part of the selected plan.

4.2 As part of the original 60 percent design submittal, a detailed review of the channel shear stresses was performed to review the rip rap and cellular protection limits. As part of the GRR, the anticipated rip rap protection was proposed to extend to 4 feet above the channel invert with cellular material placed above this elevation. Based upon the shear stress analysis, the rip rap height was adjusted to a lower elevation based upon the shear stresses near or about 1 lbs/ft^2. This value was considered an acceptable value for the use of cellular protection.

4.3 Additionally, some thought was originally given to removing the cellular bank material for cost purposes and utilizing the native natural grasses as channel erosion protection. Although this method could reduce the project construction cost, the reliability and complete coverage could not be guaranteed without a proposed irrigation system and a robust maintenance plan. Additionally, the channel side slopes would need to be regraded to a 4:1 (or 3:1 max) side slope to minimize rilling, promote growth of vegetation, and permit maintenance of the side slope. Due to ROW limitations and access road requirements flatter side slopes could not be obtained. Additionally, the calculated shear stresses showed that channel banks could be susceptible to bank erosion during large flood events where the shear stresses exceeded the permissible values for native grasses. For these overriding reasons, it was decided to keep the cellular bank protection and the benefit it provides.

4.4 Due to cost concerns, the limits of the cellular bank stabilization and protection were revised by the design team. The cellular bank material would be placed partially up the slope to a level where natural native grasses would be utilized to provide erosion protection. Although this could lead to more maintenance from erosion repairs during high flow events or rilling of the slopes, the design team preferred the limited use of cellular bank material. Details regarding determination of allowable shear stresses are provided in Section 5 and Appendix B.

Figure 4.1 Selected Plan Alignment

4.5 Upon coordination and processing of the 60% plans through the San Francisco Bay Regional Water Quality Control Board (Water Board), it was apparent that they were opposed to the use of cellular bank material as part of the project's erosion protection measures. The Water Board requested more natural protection measures. Due to the high velocities and shear stresses within the channel, simply removing the cellular bank protection is not an acceptable solution. In consultation with the Project Delivery Team (PDT), the design was revised to having three sections of erosion protection: First would be the original buried rip rap revetment that primarily provides toe protection. This rip rap toe revetment was adjusted to a lower elevation based upon the shear stresses near or about 2 lbs/ft^2. Second is a section of buried rip rap revetment—with smaller rock than the first section—in lieu of the cellular bank protection. Since the smaller rip rap section can withstand higher shear stress than cellular bank material, the limits of the second section now extend downward of the original rip rap toe revetment as well as higher along the bank than the location of the original rock/cellular material interface. The third section consists of grasses that are able to withstand the shear stresses at the upper portions of the banks. Temporary erosion protection will be provided during the first approximately three years after construction through the use of a bio-degradable erosion control fabric (coir roll) that will help to increase the erosion resistance during the establishment period of hydro-seeded native grasses and a wetland seed mix.

4.6 Additional comments from the Water Board consist of a request to replace the Los Coches Creek and Piedmont Creek RCB culverts with free spanning bridges. This option was discussed with the PDT and determined to be impractical due to the high velocities and shear stresses in this area. The existing channels are vulnerable to scour and erosion that would expose an existing sewer line at Los Coches and erode the left bank at Piedmont Creek. The use of a clear span would not provide the invert stabilization needed to prevent further erosion in these areas. The 15" sewer line crossing at each location causes a 2 foot drop into the main channel. This drop greatly increases turbulence, channel velocities, and erosion potential. Therefore, the culvert design protects the existing sewer lines and modifies the confluence angle from 90 degrees to 30 degrees so that the hydraulic design conforms to USACE requirements. The use of the culverts and revised confluence angles reduces erosion issues and results in a lower water surface (approximately 1.5 feet), thus limiting the need for additional floodwalls and other structural/concrete improvements.

4.7 Per coordination with SCVWD, the preferred width for access roads is 18 feet, and the preferred construction material is a rock aggregate base. Due to ROW limitations, the access road widths are reduced to 15 feet or less and in some locations are only provided on one side of the channel.

REACH-BY-REACH CREEK IMPROVEMENTS

The improvements described below are segmented between successive bridge crossings. The stations provided in the descriptions are based upon the revised stationing as part of the 90 percent design. The stationing of the 90 percent design was revised to join with SCVWD Lower Berryessa Creek Improvements (see Appendix H) and is different from the stationing provided in the GRR.

Interstate 680 Bridge (Station 203+00)

4.8 The I-680 Bridge marks the upstream extent of the project. Debris has accumulated at the downstream face of the bridge. This debris should be removed regularly to ensure that the condition does not produce higher than anticipated water surface elevations along the channel banks downstream of the bridge. No modifications are proposed for the bridge except for maintenance to be performed by the local sponsor.

Channel Reach from I-680 to Montague Expressway (Station 201+50 to Station 167+03)

4.9 Channel improvements consist of excavating an 8-foot to 12-foot-deep, 16-foot bottom width earthen channel up to Station 191+80 with buried rip rap and biodegradable erosion protection at 2H:1V side slopes. Along this segment one 18-foot-wide aggregate base maintenance road is provided along the right bank due to the limited ROW. Minor grading along the left bank will be performed to install the rip rap toe protection and the biodegradable erosion protection. Additionally, the existing trees located along the left bank will be protected in place.

4.10 The access road along the left bank will extend along its existing location from approximate station 167+00 to 180+00, where the channel abuts existing residential housing and there is limited existing ROW. Discussions with the project team and local sponsor determined that the additional cost to relocate the channel would add unnecessary construction costs and require the replanting of numerous existing trees along the left bank. These impacts could be avoided by the addition of three down ramps, located at approximate station 190+00 and 179+00, which would allow for routine maintenance of the left bank by utilizing the channel invert. Since this section of the channel is routinely dry, the use of the channel invert is an acceptable alternative and reduces project cost. The trees along this reach would also be protected in place and grading would be limited to outside of the canopy as determined by an arborist.

4.11 Due to the construction of an existing building and retaining wall along the left bank (Sta. 171+00 to 175+50) a buried floodwall will be provided along the channel side of the left bank access road. The floodwall is required because soil support from the privately owned retaining structures (such as the existing building and retaining wall) cannot be utilized, since their future condition could not be assured. The construction of the buried floodwall will eliminate this reliance on the existing building and retaining wall. An aggregate base access road with a minimum width of 10 feet is provided for maintenance.

4.12 Upstream of Station 191+00, debris has accumulated along the channel invert; this should be removed regularly to ensure that the condition does not produce higher than anticipated water surface elevations along the channel banks. No modifications are proposed for the channel except for routine/regular maintenance to be performed by the local sponsor and the addition of a 20-foot section of rip rap channel downstream of the concrete channel at Sta. 199+00. Additionally, existing trees located along the right bank will be protected in place.

Montague Expressway Bridge (Station 166+00)

4.13 Montague Expressway is currently under construction by the Valley Transportation Authority (VTA) in preparation of the BART Milpitas station. The proposed improvement include a nine-lane arterial crossing over the Berryessa Creek and replacement of the existing double-barrel 12-foot by 10-foot culvert with a clear span bridge section. The increased capacity from the improvements eliminate the need for above ground floodwalls upstream of Montague Expressway. The proposed construction of these improvements will be performed concurrently with the Berryessa Creek project and traffic staging will need to be coordinated along with the construction of the channel improvements. The channel improvements will join the proposed concrete side slope with rip rap bottom improvements proposed by the bridge improvements.

Channel Reach from Montague Expressway to UPRR Trestle (Station 161+25 to Station 164+93)

4.14 Channel improvements consist of excavating a 10.5-foot-deep, 12-foot bottom-width earthen channel with buried rip rap and biodegradable erosion protection at 2H:1V side slopes. Two aggregate base maintenance roads, 18-foot-wide and 15-foot-wide, are provided on the right and left banks, respectively. Since the bottom width is 12 feet, the channel invert will be lined with buried rip rap section in lieu of providing toe down protection for each bank.

UPRR Railroad Trestle Bridge (Station 161+00)

4.15 The existing UPRR trestle is a timber railroad crossing with four sets of piers. Due to the condition of the existing structure, excavation around the bed or banks was deemed unacceptable and complete replacement of the trestle is proposed. A double barrel 10-foot-wide by 9-foot-high reinforced concrete box culvert would be installed. The culvert will be pre-cast. New railroad tracks will be reconstructed on top of the new double barrel box culvert along with new ballast rock, rails, and wooden ties. The culvert would also support vehicular access to the east bank of the creek (Sta 144+00 to 160+00). The access road would run parallel to the rail road tracks (without crossing the tracks) and provide vehicular access to east bank.

Channel Reach from UPRR Trestle to UPRR Culvert (Station 160+75 to Station 142+93)

4.16 Channel improvements consist of excavating a 9-foot-deep to 13-foot-deep, 12-foot bottom-width earthen channel with buried rip rap and biodegradable erosion protection at 2H: 1V side slopes. Two aggregate-base maintenance roads, 18-foot-wide and 15-foot-wide, are provided on the right and left banks, respectively. Since the bottom width is 12 feet, the channel invert will be lined with buried rip rap section in lieu of providing toe down protection for each bank.

UPRR Railroad Culvert (Station 142+00)

4.17 The channel transitions to a wider available ROW where Milpitas Boulevard veers away from the channel upstream of the UPRR culvert. The existing structure has sufficient conveyance to meet the requirements of the selected plan provided the channel banks are tied into the existing concrete wing walls.

Channel Reach from UPRR Culvert to Ames Avenue (Station 141+60 to Station 137+70)

4.18 Channel improvements consist of excavating an 11-foot-deep, 12-foot bottom-width earthen channel with buried rip rap and biodegradable erosion protection at 2H: 1V side slopes. An 18-foot-wide aggregate base maintenance road is provided along the right and left banks.

Since the channel bottom width is 12 feet, the channel invert will be lined with buried rip rap section in lieu of providing toe down protection for each bank.

Ames Avenue Bridge (Station 137+50)

4.19 Ames Ave is a two lane bridge, approximately 63-feet long consisting of two spans supported by an interior pier. The bridge spans are approximately 30-feet each for a total length of approximately 60 feet. However, the existing ground blocks much of the cross section below the bridge deck. This ground will be excavated to allow for construction of the channel improvements. As part of the original design, the bridge design assumed this ultimate graded channel section beneath the bridge.

4.20 Abutment and pier scour protection is proposed for the bridge. The design will protect the piers/abutments from increased flood volumes and velocities with the addition of rip rap protection to mitigate for the potential undermining that could result from bridge scour caused by the increased channel capacity.

4.21 Below the bridge, the proposed channel improvements will consist of excavating a 12-foot deep, 12-foot bottom width buried rip rap channel with 2H: 1V side slopes. The two 18-foot aggregate base maintenance roads will be graded and aligned to join the existing driveways provided along the street.

4.22 Along the upstream right bank, a six foot section of the bridge railing will be removed to allow for the connection of the access road to Ames Avenue. The existing bridge railing is a nonstructural section and will not affect the bridge deck.

Channel Reach from Ames Avenue to Yosemite Drive (Station 137+05 to Station 124+53)

4.23 Channel improvements consist of excavating a 9.5-foot-deep, 12-foot bottom width earthen channel with buried rip rap and biodegradable erosion protection at 2H: 1V side slopes. An 18-foot aggregate base maintenance road is provided on the right and left banks. Since the bottom is 12 feet wide, the channel invert will be lined with buried rip rap section in lieu of providing toe down protection for each bank.

4.24 An existing 15-inch sewer line (owned by City of Milpitas) is located along the right overbank bank and will be protected, in place, during construction.

Yosemite Drive Bridge (Station 124+15)

4.25 The Yosemite Drive Bridge is a four lane bridge, approximately 75-feet long consisting of two spans supported by an interior pier. The bridge spans are approximately 30-feet each for a total length of approximately 60 feet. A major pipeline is supported by cantilevered structural elements along the upstream face of the bridge. The existing ground blocks much of the cross section below the bridge deck. This ground will be excavated to allow for construction of the channel improvements. As part of the original design, the bridge design assumed this ultimate graded channel section beneath the bridge.

4.26 Abutment and pier scour protection is proposed for the bridge. The design will protect the piers/abutments from increased flood volumes and velocities with the addition of rip rap protection to mitigate for the potential undermining that could result from bridge scour caused by the increased channel capacity.

4.27 Below the bridge, the proposed channel improvements will consist of excavating a 12-foot-deep, 20-foot bottom width buried rip rap channel with 2H: 1V side slopes. The 18-foot-wide aggregate base maintenance roads will be graded and aligned to join the existing driveways provided along the street.

4.28 The existing City of Milpitas waterlines along the bridge soffit and below the channel will be protected in place during construction and will not be impacted.

Channel Reach from Yosemite Drive to Los Coches Street (Station 123+80 to Station 93+25)

4.29 From Los Coches Street Bridge to Piedmont Creek confluence (Station 93+25 to Station 115+23), the channel improvements consist of excavating a 9- to 14-foot-deep, 40-foot bottom-width earthen channel with buried rip rap and biodegradable erosion protection at 2H:1V side slopes. A 15-foot-wide and 18-foot-wide aggregate base maintenance road is provided on the right and left bank, respectively. A small floodwall is provided along the right bank from Station 105+00 to Station 116+23 (1,123 linear feet) to maintain a minimum channel depth of 9.5 feet.

4.30 From the Piedmont Creek confluence to Yosemite Drive Bridge (Station 115+23 to Station 123+80), the channel improvements consist of excavating a 10- to 13.5- foot-deep, 20-foot bottom-width earthen channel with buried rip rap and biodegradable erosion protection at 2H:1V side slopes. An 18-foot-wide aggregate base maintenance road is provided on the right and left banks. Since the bottom width is 20 feet wide, the buried rip rap toe protection will continue along the bottom of the channel. Within this section, two groundwater extraction vaults along the right bank will be protected in place.

4.31 The Piedmont Creek earthen channel will be modified to transition into a 14-foot-wide by 6-foot-high reinforced concrete box culvert (RCB) culvert. The confluence angle will also be modified from the existing 90-degree confluence to approximately 30 degrees to improve the channel hydraulics. The construction of the RCB culvert will allow for the continuation of the right bank access road across the creek. Since the existing UPRR Bridge was determined to be in disrepair, the RCB is proposed to extend upstream of the existing bridge with a transition structure to intercept the channel flows. By extending the culvert upstream of the existing bridge, the transition to a smaller size RCB (single cell instead of a double cell 14 feet wide) can be utilized. In addition, the alignment allows for a smoother transition and junction with Berryessa Creek. In all, the revised junction provides a lower water surface (approximately 1.5 feet) over the previous design and allows for a reduction in floodwalls and other channel improvements. This design modification provides a net benefit to the project without any additional construction cost and thus was incorporated into the design.

4.32 An existing 15-inch sewer line (owned by City of Milpitas) along the right bank will be protected in place during construction. In addition, an existing PG&E electrical vault will be protected in place. The existing City of Milpitas exercise pocket park will be removed to allow for the construction of the 18-foot-wide access road. The City also proposing to install two new waterline crossings at Los Coches Street. These improvements will be coordinated with the project so that the portions crossing the sewer could be installed prior to the construction of the concrete transition structures. No utility relocations will be required through this reach.

Los Coches Street Bridge (Station 93+00)

4.33 The Los Coches Street Bridge is a two lane bridge, approximately 48-feet long consisting of two spans supported by an interior pier. The bridge spans are approximately 37-feet each for a total length of approximately 74 feet. The left side of the channel is concrete, while the right side is earthen. The Arroyo de los Coches tributary enters at the upstream face on the right bank. The existing structure allows for sufficient conveyance to accommodate the selected plan, provided the channel walls are tied into the existing structure. Directly upstream of the Los Coches Street Bridge is a free-span pedestrian bridge.

4.34 Directly upstream of the Los Coches Street Bridge crossing, a 50-linear-foot concrete transition structure is provided, which transitions the 40-foot-wide-bottom trapezoidal channel to a 60-foot-wide rectangular channel section. The transition will be off center to allow for the realignment of the Arroyo de los Coches channel confluence.

4.35 The Arroyo de los Coches earthen channel will be modified to transition into a 14-foot-wide by 6-foot-high RCB culvert. The confluence angle will also be modified from the existing 90-degree confluence to approximately 30 degrees to improve the channel hydraulics. The construction of the RCB culvert will allow for the continuation of the access road to connect to Los Coches Street.

4.36 Below the bridge, a concrete rectangular channel with a 75-foot bottom-width will be provided to convey the Berryessa Creek and Arroyo de los Coches storm flows. The concrete rectangular channel will be designed to protect the existing bridge abutments.

4.37 A 50-foot concrete transition structure is provided downstream of the bridge to assist in the conveyance of storm flows. This will transition the channel back to a 40-foot-wide-bottom trapezoidal channel

4.38 The existing pedestrian bridge will be protected in place as the proposed improvements will avoid impact to the bridge or the abutments.

4.39 An existing 15-inch sewer line (owned by City of Milpitas) will be protected in place during construction. No utility relocations will be required through this reach.

Channel Reach from Los Coches Street to Calaveras Boulevard (Station 87+20 to Station 92+70)

4.40 Channel improvements consist of excavating a 12- foot-deep to 14-foot-deep, 40-foot bottom width earthen channel with buried rip rap and biodegradable erosion protection at 2H: 1V side slopes. A 15-foot and 18-foot aggregate base maintenance roads are provided on the right and left bank, respectively.

4.41 An existing 18-inch sewer line (owned by City of Milpitas) will be protected in place during construction. There is an existing sampling/gauging station located at approximately Station 89+00 that will need to be removed and replaced (by others) to allow for construction of the channel improvements. No additional utility relocations will be required through this reach.

Calaveras Boulevard Bridge (Station 86+50)

4.42 The Calaveras Boulevard Bridge culvert is an eight-lane divided roadway. The crossing is comprised of four, 8-by-11-foot culvert barrels. The outer two barrels are partially filled with the earthen side slopes that project into the outside toe of the middle culvert barrels. Debris has accumulated to about one to two feet high within the inner two barrels.

4.43 The existing bridge provides sufficient conveyance to accommodate the selected plan, provided the sediment in the outer barrels is removed, and the channel walls are tied into the existing structure. A 50-foot concrete transition is provided at the upstream end to assist in the conveyance of storm flows.

Channel Reach Downstream of Calaveras Boulevard (Station 131+05 to Station 129+80)

4.44 The channel downstream of Calaveras Boulevard will be widened and improved based on SCVWD's Lower Berryessa Creek channel improvements. (See Appendix H for the 90 percent design plans.) This is a separate project administered by Santa Clara Valley Water District.

CONSTRUCTION STAGING

4.45 The Upper Berryessa Creek Flood Management Project may require construction staging to avoid increasing flood risks downstream, while adhering to the project schedule. Hydraulic analyses were performed to identify possible staging opportunities. The recommended sections for construction extend from 500' upstream of Piedmont Creek confluence to just downstream the UPRR trestle bridge, and from downstream I-680 to 1,200' upstream of Montague Expressway. The analysis performed by SCVWD is provided in Appendix B.

UNION PACIFIC RAILROAD TRESTLE BRIDGE REPLACEMENT (Station 161+00)

4.46 The project team has coordinated with Union Pacific Railroad (UPRR) in the design of the trestle bridge replacement. An initial project meeting with UPRR occurred on December 4, 2014 at their local Roseville office.

4.47 In general, UPRR's preference is to build a new creek crossing and then demolish and remove the trestle, which will minimize the amount of time the track is out of service. A permanent realigned crossing rather than a temporary shoofly would be preferred, if possible. If a shoofly/alternate alignment is not possible, then a plan for weekend work will be necessary where a precast box culvert is dropped in and track placed with one to three days track downtime. UPRR crews will install and reconnect the track after the culvert work is finished, and can complete this work in a short period of time.

4.48 Based upon this meeting, an alignment study was performed which showed the impact of a track re-alignment and the required ROW acquisition (See Figure 4.2). The design of the shoofly would require the following additional costs:

- Railroad crossing signal installation (includes removal of temporary crossing signals) - $650,000
- Track construction (includes removal of temporary tracks within the roadway) - $300,000
- Roadway modifications (includes removal of temporary paving) - $200,000
- Additional cost associated with encroaching within adjacent private property and permits with the County/City covering realignment of UPRR tracks.

4.49 Due to the high relocation cost to build a shoofly track, the selected design option will plan for an extended weekend work period when the track can be shut down. During this long weekend, a precast box culvert will be placed across the Berryessa Creek channel, and UPRR crews will install and reconnect the track after the culvert work is finished.

4.50 During the 90 percent design, the project team coordinated with UPRR engineering and operations staff to obtain a conceptual approval for the plan that has been discussed to remove/replace the trestle/culvert during a two- to three-day period. A general concept plan was provided and is reflected in the current 90 percent design plans. Detailing on the removal of the existing trestle and insertion of the box culvert will occur after further coordination with UPRR and will be included in the 90 percent plans.

4.51 During construction, UPRR or its construction contractor will perform all of the removal and placement of the track, ballast, and rail road ties. The Berryessa creek contactor will construct all other items including the excavation and foundation preparation.

4.52 A Construction and Maintenance (C&M) agreement will need to be finalized prior to construction to identify what costs UPRR will be reimbursed for related to the culvert/tracks, and what maintenance issues there will be in the future, if any, requiring compensation.

4.53 A review reimbursement agreement with UPRR has been executed. The agreement requires the project to pay up to $25,000 for reimbursement for review of the plans; the payment only occurs at the end of the review process. The Corps will be financially responsible for installing the new railroad crossing of the creek, as railroad bridges are not covered in LERRDs.

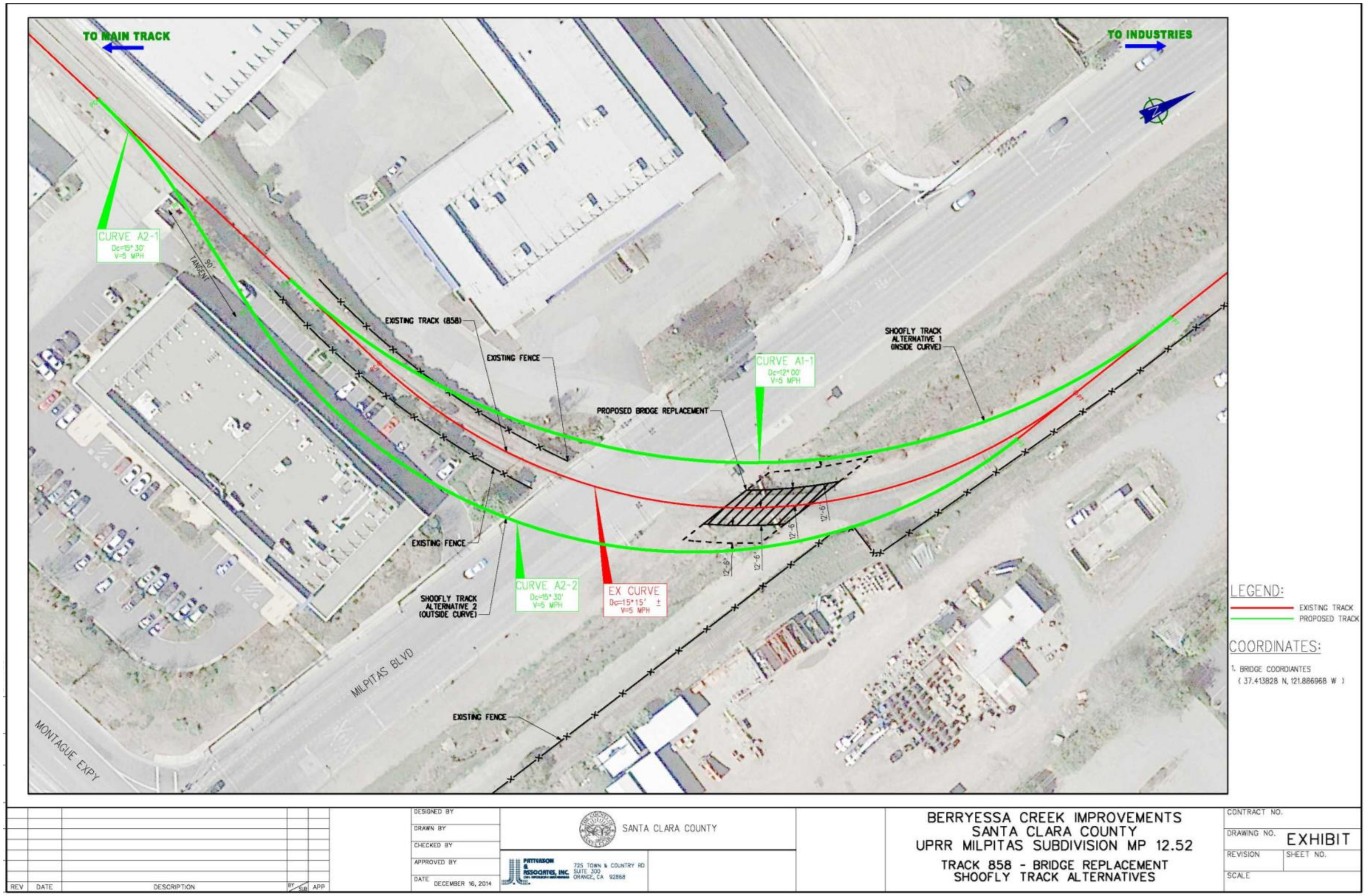

Figure 4.2 Shoofly Alternatives

STOCKPILING OF TOPSOIL

4.54 In the design of the channel, the rip rap inverts are proposed to be choked with the native bed material consisting of sands and gravels. To accomplish this approximately 3,000 CY of bed aggregates will need to be excavated and stockpiled. The stockpiling of material will occur within the project limits or staging areas with in accordance with the SWPPP for the project. Additionally, the bank improvements propose choking the rip rap revetment with native bank material to allow for the growth of vegetation over the rip rap revetment. Since the bank material is readily available and consistent throughout the project large stockpiles of bank material is not expected. The project will require export and disposal of excess soils to a suitable location. The contractor will be responsible for exporting and transport of the excess soil off the project site.

MATERIAL REQUIRED FOR CONSTRUCTION

4.55 Material required for project construction will include earth fill material; concrete for walls, footings, and box culvert; rocks/rip rap for slope protection; steel reinforcement for concrete support; filter material; fencing material; and top soil. Except for the rocks/rip rap, most of the material will be obtained from a distance of about 5- to 10-mile radius from the project area. Rip rap may be obtained from existing quarries located within 50 miles.

4.56 The construction of the channel requires material to be excavated and hauled off-site for disposal or stockpiled onsite and reused for fill. An expansion and shrinkage factor of 25% was applied for the grading volumes calculated by 3-D computer models. The grading models compared the existing surface with the proposed post project surface to generate excavation and fill volumes for the project. These quantities are augmented with hand calculations to account for the volumes generated by the placement of rip rap, excavation to construct the rip rap protection, and the floodwalls. Based upon the grading calculations the project expects to excavate and haul approximately 148,400 CY of channel soil. Of this material approximately 41,900 CY will be utilized for fill and approximately 114,500 CY will be disposed by the contractor offsite. The detailed calculations are provided in the cost estimate report (Appendix F).

HAZARDOUS AND TOXIC MATERIALS AND WASTE

4.57 Seven sites of interest have been identified and considered serious existing situations by the RWQCB. These sites are discussed in the following sections.

4.58 The Great Western Site was a chemical depot and distribution business in operation between the late 1950s and the mid-1980s. Past operations included chemical storage in four 6,000-gallon and other smaller above-ground storage tanks (ASTs), and eight 7,500-gallon underground storage tanks (USTs). Remediation actions conducted during many years have focused on improving groundwater quality and controlling and reducing off-site migration of impacted groundwater from the on-site source zones. Accidental releases and operational procedures during the life of the Great Western Facility have resulted in high concentrations of VOCs found – through ongoing investigations and monitoring – to be present in the groundwater under this site, and through down-gradient migration from this site as an underground "plume" of VOC-contaminated groundwater. Although available reports have not provided a precise boundary of the Great Western plume, based on available information, the plume can reasonably be considered to be underlying what is referred to as the "Off-Site Area." Berryessa Creek crosses the southeastern portion of the off-site area of the Great Western Site.

4.59 Based on the positive results in reducing VOC levels at the downstream contaminant plume (Great Western Site), the RWQCB approved a proposal in October 2012 to close further remediation efforts and destroy the wells associated with the remediation and monitoring related to the GW plume. The closure was completed by the end of 2012.

4.60 The JCI Site was also a chemical storage and distribution business, in operation between the early 1960s and the late 1990s. The JCI Site routinely received (by rail or tank truck) and stored chlorine gas, sulfur dioxide, anhydrous ammonia, various acid and bases, as well as trichloroethane (TCA). Based on the nature of contamination on the JCI Site and the hydrogeological characteristics of the underlying sediments resulting in off-site migration of pollutants, ongoing remediation actions, monitoring, and investigations have been conducted both on-site and off-site. The remediation actions have focused on improving groundwater quality and controlling and reducing migration of impacted groundwater. The installation and ongoing use of numerous extraction, injection, and monitoring wells in the on-site and off-site areas have been associated with the remediation actions. Groundwater levels have also been periodically monitored in selected wells for both the on-site and off-site areas. Berryessa Creek passes adjacent to the west boundary of the JCI Site. The HTRW evaluation determined that groundwater will need to be retained during construction in this area if dewatering is performed**.** A groundwater management plan has been prepared for treatment of groundwater if encountered. Further details and information regarding the treatment of the groundwater from this site is provided in Section 9 of the DDR.

4.61 The Penske Truck Leasing Site (Penske Site) was formerly operated as a fleet rental, servicing, repair, and fueling operations facility until September 2003. Former features at the Penske Site included two 20,000-gallon diesel fuel USTs, one 500-gallon waste oil AST, one 1,500-gallon new oil AST, and four dispenser islands. All of these features were removed in 2003. Soil testing at the Penske Site taken at the time, and groundwater samples taken in 2004 indicated the presence of TPH-d and TPH-g in the soil and groundwater that were above ESLs. Berryessa Creek is located approximately 500 feet west and down gradient of the removed features.

4.62 This North American Transformer (NAT) Site was formerly used as a manufacturing, testing, and repair facility for electrical transformers from about 1958 to 2002. The NAT Site is located about 1,200 feet west and down-gradient of Berryessa Creek. Remediation efforts to bring VOCs in the groundwater under the NAT Site to within acceptable ecological screening levels (ESLs) are ongoing, through the efforts of the JCI Jones Site, and under the oversight of the RWQCB.

4.63 The Linear Technology Corporate Site is located about 500 feet west and down-gradient from Berryessa Creek. Information on this site is marginal. The RWQCB GeoTracker database does not have any data or information on the site.

4.64 The Lite-On Inc. Site is located about 100 feet west and adjacent to Berryessa Creek. Information on this site is marginal. The RWQCB GeoTracker database does not have any data or information on the site.

4.65 The DISC Stampers LLC Site is located about 500 feet west and up-gradient from Berryessa Creek, in the same vicinity as the Penske Site. Information on this site is marginal. The RWQCB GeoTracker database does not have any data or information on the site.

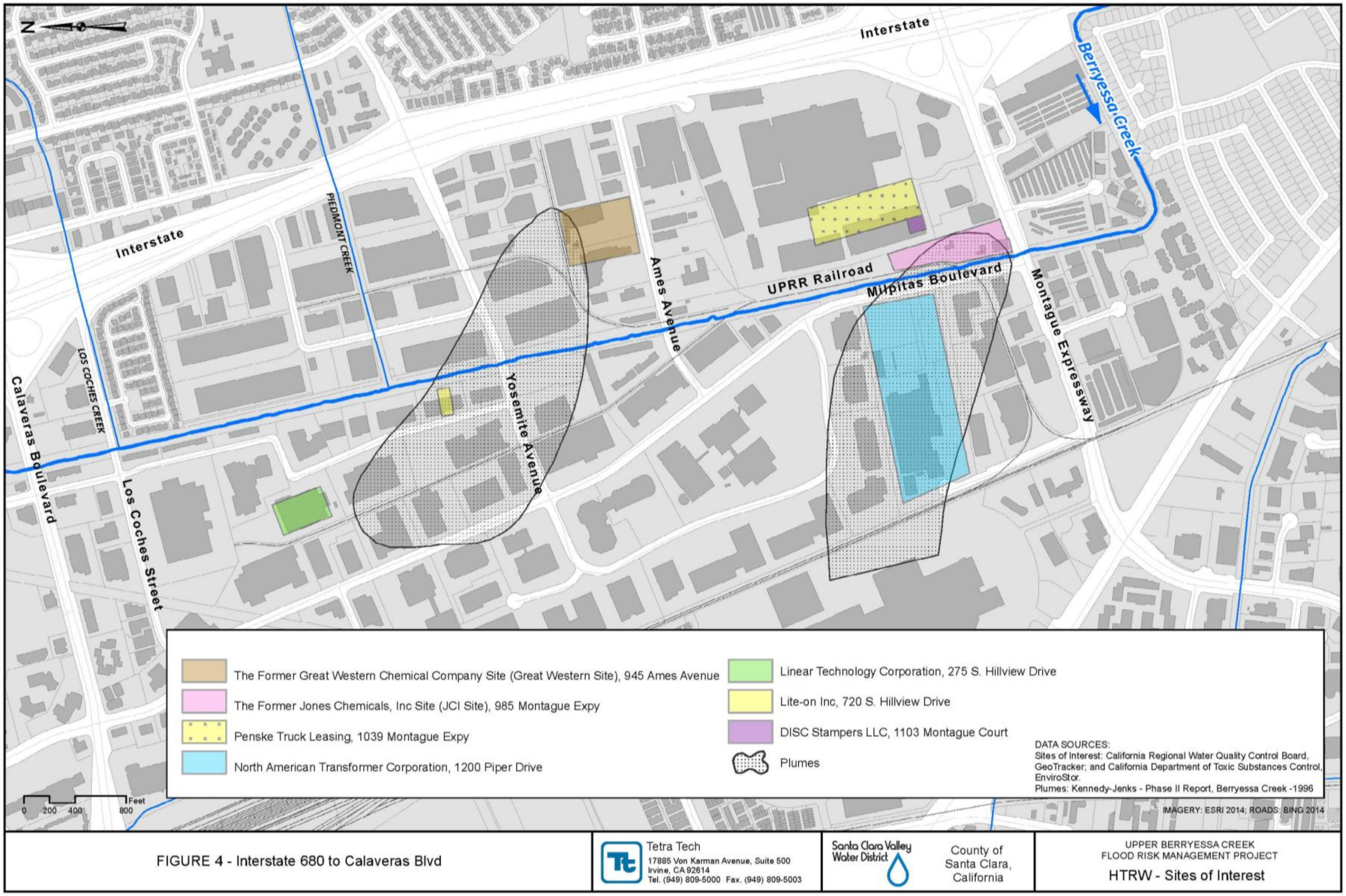

Figure **4.3** **HTRW Sites of Interest**

5. HYDROLOGIC AND HYDRAULIC BASIS FOR DESIGN

5.1 This section describes the hydraulic analysis that was conducted to support the project design. The analysis described here includes hydraulic modeling of the baseline (existing) and with-project (proposed) conditions. Descriptions of the assumptions, inputs, methodologies, and results of this analysis are provided below.

5.2 The HEC-GeoRAS program (version 10.1 for ArcGIS 10.1) was used to develop geo-referenced stream station lines, cross section alignments and cross section profiles. The cross section profiles were cut using the electronic three-dimensional surface for the portion of the reach from upstream of I-680 to downstream of Calaveras Boulevard. The baseline geometry was based on GeoRAS-extracted cross-sections from 1-foot contour interval topography. The with-project condition geometries downstream of I-680 and upstream of Calaveras Boulevard were modeled based on an updated version of Alternative 2A from the final GRR/EIS (USACE 2014). The development of the baseline and with-project geometries is further defined in the following sections.

5.3 The baseline conditions model was truncated at the Alternative 2A improvement reach. This reach extends along Berryessa Creek bounded by I-680 on the upstream end and just below the confluence with Tularcitos Creek on the downstream end. Modeling was completed using HEC-RAS v 4.1.0 a steady state flow simulation was developed as follows:

- For the Baseline Model, new cross-sections were extracted from the new topographic mapping (Figure 5.1). Cross section spacing and locations were generally maintained to be consistent with the GRR model. Typical spacing averaged approximate 100-foot intervals, and extended approximately 150 feet wide to model the primary channel and align with the updated topographic mapping.
- The downstream boundary condition was based on a rating curve.
- Upstream boundary condition was based on critical depth.
- Bridge openings and culvert sizes maintained the GRR model dimensions.
- Routed peak flows were based upon the GRR model hydrology (NHC 2006). HEC-HMS model results (NHC 2006) were augmented with a FLO-2D model upstream of the I-680 in the final GRR/EIR (USACE 2014) and those routed hydrographs were utilized in the unsteady baseline condition model.

5.4 This hydraulic model will be the baseline model from which the new Alternative 2A model is developed and to further refine the Berryessa Creek Design. A new centerline and stationing to match SCVWD Lower Berryessa Creek design was incorporated into the refinement of the Alternative 2A design. Because of the revised centerline stationing, the HEC-RAS model sections from the GRR model do not coincide with the River Stations developed for the Alternative 2A model. Table 5.1 shows how the stationing of the two models compare.

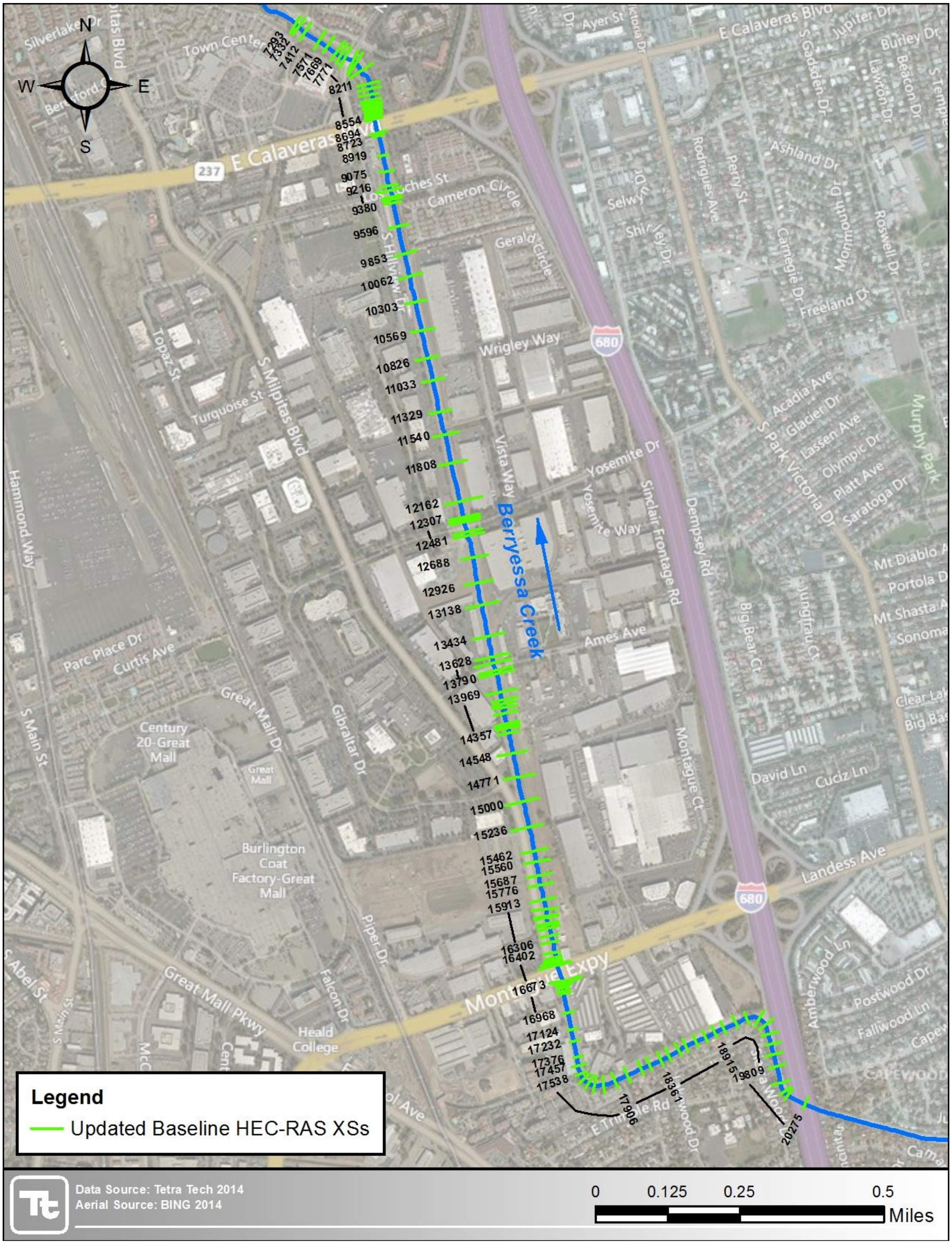

Figure 5.1 Updated Baseline HEC-RAS Cross-Sections

5.5 Utilizing the unsteady baseline hydraulic model, which routes the HEC-HMS model results (NHC 2006) with the augmented with a FLO-2D model upstream of the I-680, key points were identified where flow changes occurred and the resulting peak flow rates are utilized in the steady state Alternative 2A model for the advancement of the design. Hydrologic input locations based on the new centerline stationing are shown in Table 5.1 for both the baseline and the 60 percent Alternative 2A design models.

Table 5.1 Baseline and Design Model Hydrologic Input River Stations

RAS River Station		HEC RAS Discharges							
Baseline	**Alt 2A**	**500-yr**	**200-yr**	**100-yr**	**50-yr**	**25-yr**	**10-yr**	**5-yr**	**2-yr**
202+75	201+32	1770	1611	1545	1403	1145	954	700	490
143+57	143+42	2693	2445	2010	1730	1545	1224	947	620
124+81	125+19	2970	2633	2170	1870	1665	1294	1009	660
115+40	114+73	3800	3485	3112	2680	2360	1733	1379	878
92+16	92+20.1	4640	4355	3885	3365	2850	2173	1737	1130
90+75	89+95.04	5037	4750	4100	3365	2850	2173	1737	1130

5.6 Under the existing or baseline conditions, the computed 100-year water surface elevations along Berryessa Creek are shown on Figure 5.2 and depict the overtopping of Montague Expressway, Los Coches, and Calaveras Boulevard under the existing geometry conditions.

5.7 An new proposed conditions HEC-RAS Model for the with-project model was generated under the following conditions:

- Model station and downstream boundary condition was based upon the SCVWD 90 percent Lower Berryessa Creek design and hydraulic model with updated flow rates from project.
- The Montague Expressway bridge replacement as part of VTA improvements
- Individual bridges and culverts to be modified and resized in conjunction with Alternative 2A were incorporated within the model. This included the addition of concrete lined transition structures up and downstream of the bridges.
- Though the modeled inverts may have slight differences from the design plans, the general channel shape from the 60 percent plans was used in the hydraulic model.

5.8 Bridge replacement scenarios assume concrete barriers are part of the bridge deck (obstructed), while rails are not. Proposed channel excavation for increased conveyance was generally designed by spreadsheet and interpolated along like reaches. In general, design sections were developed where proposed changes in the cross-section occurred. Channel excavation templates generally follow a smooth slope between existing bridge inverts.

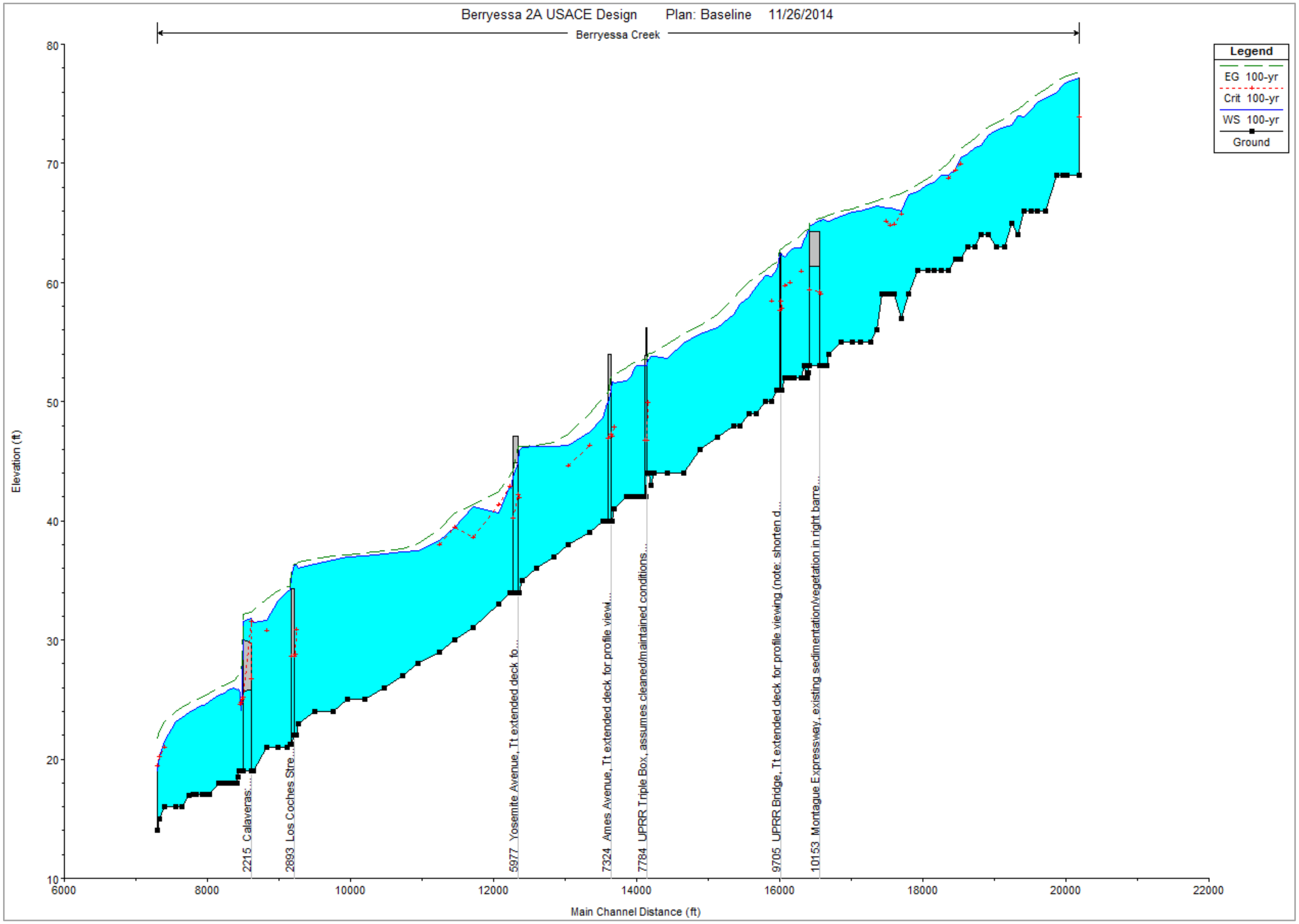

Figure 5.2 Simulated Water Surface Profile for 100-Year Flood Event (Baseline Conditions)

Table 5.2 Alternative 2A 60% Plans Design Sections

Typical Section	Berryessa Creek River Stations	Shape	Bottom Width	Side Slope (H:1V)
1	87+20 to 92+21	Trapezoidal	40	2
2	93+76 to 114+73	Trapezoidal	40	2
3	115+23 to 124+53	Trapezoidal	20	2
4	125+91 to 164+43	Trapezoidal	12	2
5	167+56 to 191+00	Trapezoidal	16	2

5.9 Under the with-project Updated Alternative 2A Design, the computed 100-year water surface profile for the updated design conditions for Alternative 2A is effectively passed under the roadways as shown on Figure 5.3.

5.10 Comparison of the results clearly shows the excavation and reshaping of the Berryessa Creek Alternative 2A reach and improved conveyance capacity of the updated design channel. A comparison profile is shown on Figure 5.4.

5.11 It should be noted that the Lower Berryessa Creek HEC-RAS model constructed by SCVWD had been appended to the downstream of Berryessa Creek Alternative 2A HEC-RAS model per SCVWD's request. A known water surface elevation of 21.07 feet in the Lower Berryessa Creek HEC-RAS model was adopted as the downstream starting water surface elevation, based on the SCVWD Memorandum, *Starring Water Surface Elevation for Lower Penitencia and Lower Berryessa Creek Project*, prepared in March 2015. This water surface elevation was derived by incorporating Mean Higher High Water or 10-year Tide at mouth of Coyote Creek and potential sea level rise at Year 2067.

5.12 Sensitivity analysis of the downstream controlling water surface elevation was conducted for three scenarios which include the possible highest water surface elevation of 21.82 feet, the SCVWD design water surface elevation of 21.07 feet (SCVWD 2015), and critical flow depth. The results indicate the maximum deviation of 0.3 feet of the computed water surface elevations occur immediately upstream of Calaveras Boulevard and the deviations diminish at approximately 500 feet downstream of Yosemite Avenue. It was concluded that the downstream controlling water surface elevation has insignificant impacts to the project reach.

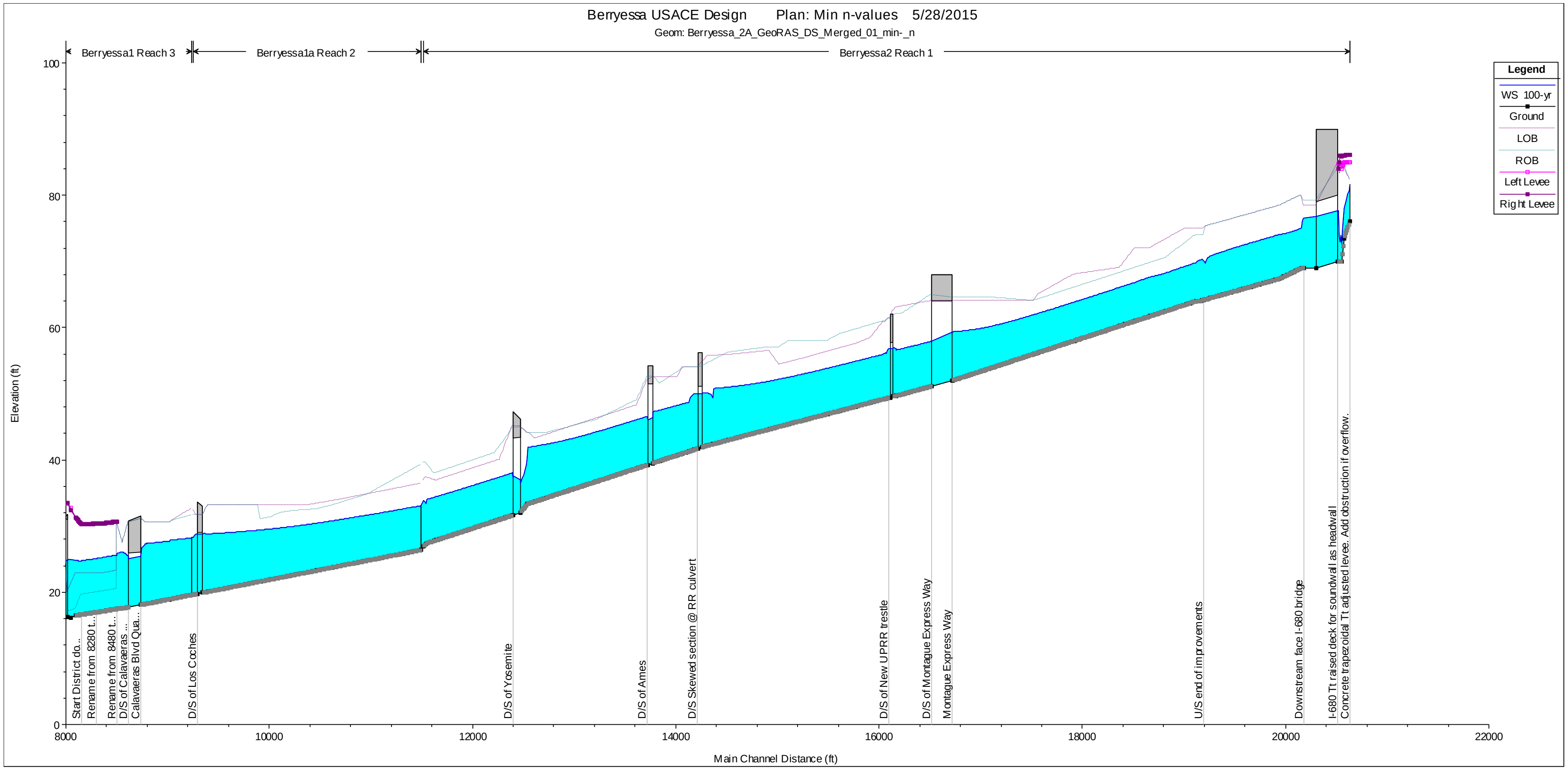

Figure 5.3 Simulated Water Surface Profile for 100-Year Flood Event (Alternative 2A Conditions)

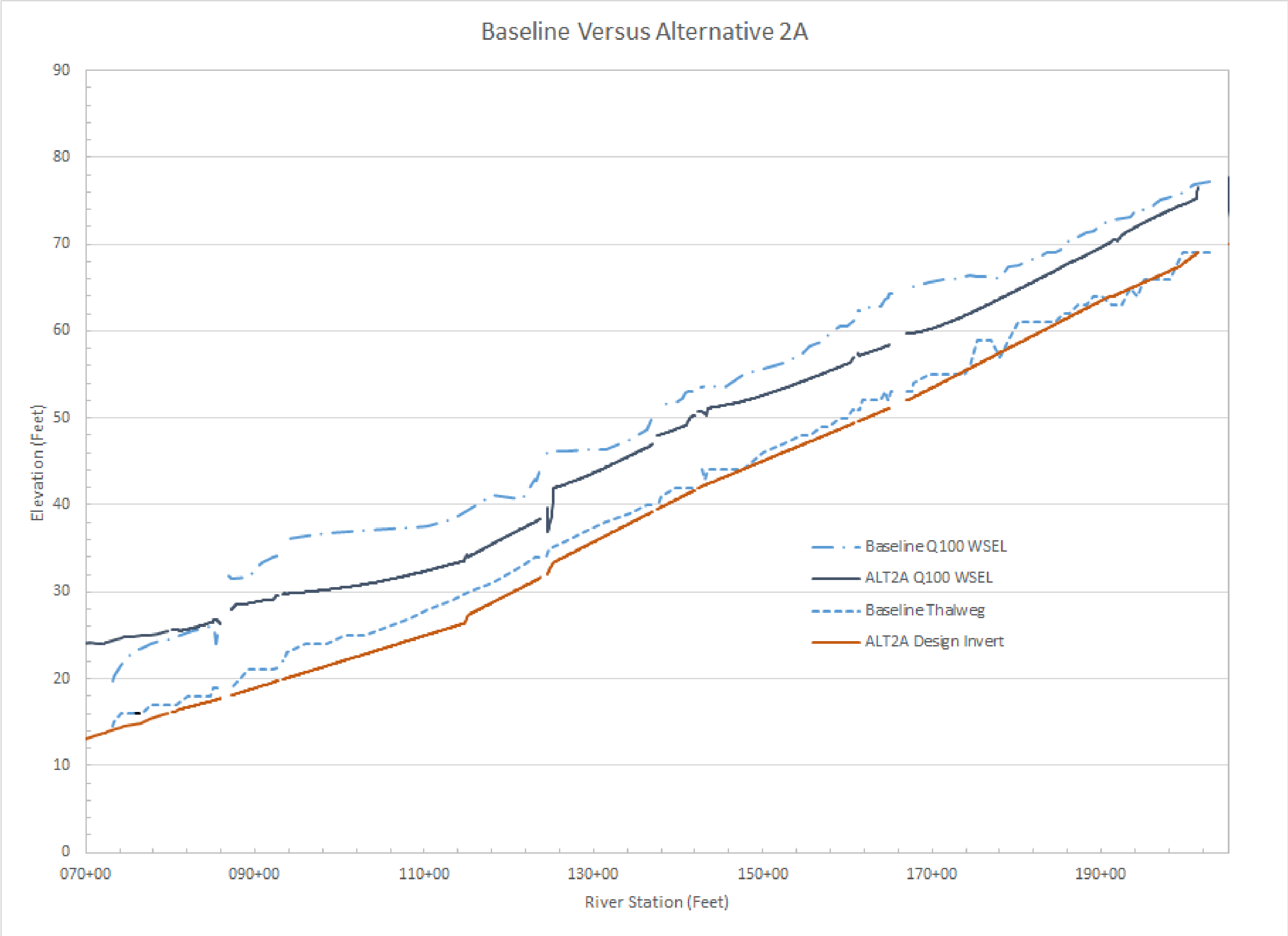

Figure 5.4 Simulated Water Surface Profile Comparison for 100-Year Flood Event

Scour Analysis

5.13 Five representative HEC-RAS segments defined by the five typical sections in the 60 percent Alternative 2A design are used in determining the single-event scour for refinements in the design. Single-event scour is normally computed as the sum of general scour, bed-form depth, low-flow incisement, local scour, and bend scour. The following paragraphs describe the estimation of each single-event scour component. Note that long-term scour [i.e., degradation] is typically computed separately and was not included in this assessment. It is assumed that long-term scour will be mitigated with routine maintenance as it develops. In general, the calculation of the individual scour components is based on the HEC-RAS hydraulic model results of flow velocity, depth, energy slope, and Froude number at each channel cross section.

Estimate of General Scour

5.14 For the general scour condition, additional sediment analysis was performed as part of the hydraulic analysis (see Sediment Transport Section). As shown in the analysis, the general scour is estimated be to less than 1 foot. Therefore, 1 foot was assumed and adopted as the general scour depth.

Bed Form Depth

5.15 For the purposes of evaluating an upper envelope for temporary scour that can occur during the passage of flood flows, differentials in streambed gradient associated with channel bed formations is considered. Bed forms are a second type of scour that can occur in sand-bed channels during a flood event. For purposes of evaluating the maximum streambed changes during the passage of a single event, two main bed forms, dunes or anti-dunes, are considered. In general, dunes typically form in lower regime flow (highly subcritical) and anti-dunes develop when flows are upper regime (at or near critical). Essential to properly characterizing the single event scour, a determination was made of the flow regime, either upper or lower. The distinction between flow regimes was made using the applicable charts found in the *Manual on Sedimentation Engineering* (ASCE 2006) and presented in Figure 5.5.

Flow regime is determined by the Froude number, F_R, as follow:

Lower regime flow if $F_R \leq 4.39 \cdot (R/D_{50})^{-0.3}$
Transition zone if $4.39 \cdot (R/D_{50})^{-0.3} < F_R < 4.949 \cdot (R/D_{50})^{-0.27}$
Upper regime flow if $F_R \geq 4.949 \cdot (R/D_{50})^{-0.27}$

Where :

F_R = Froude number
R = Hydraulic radius, in feet
D_{50} = Median particle diameter, in feet

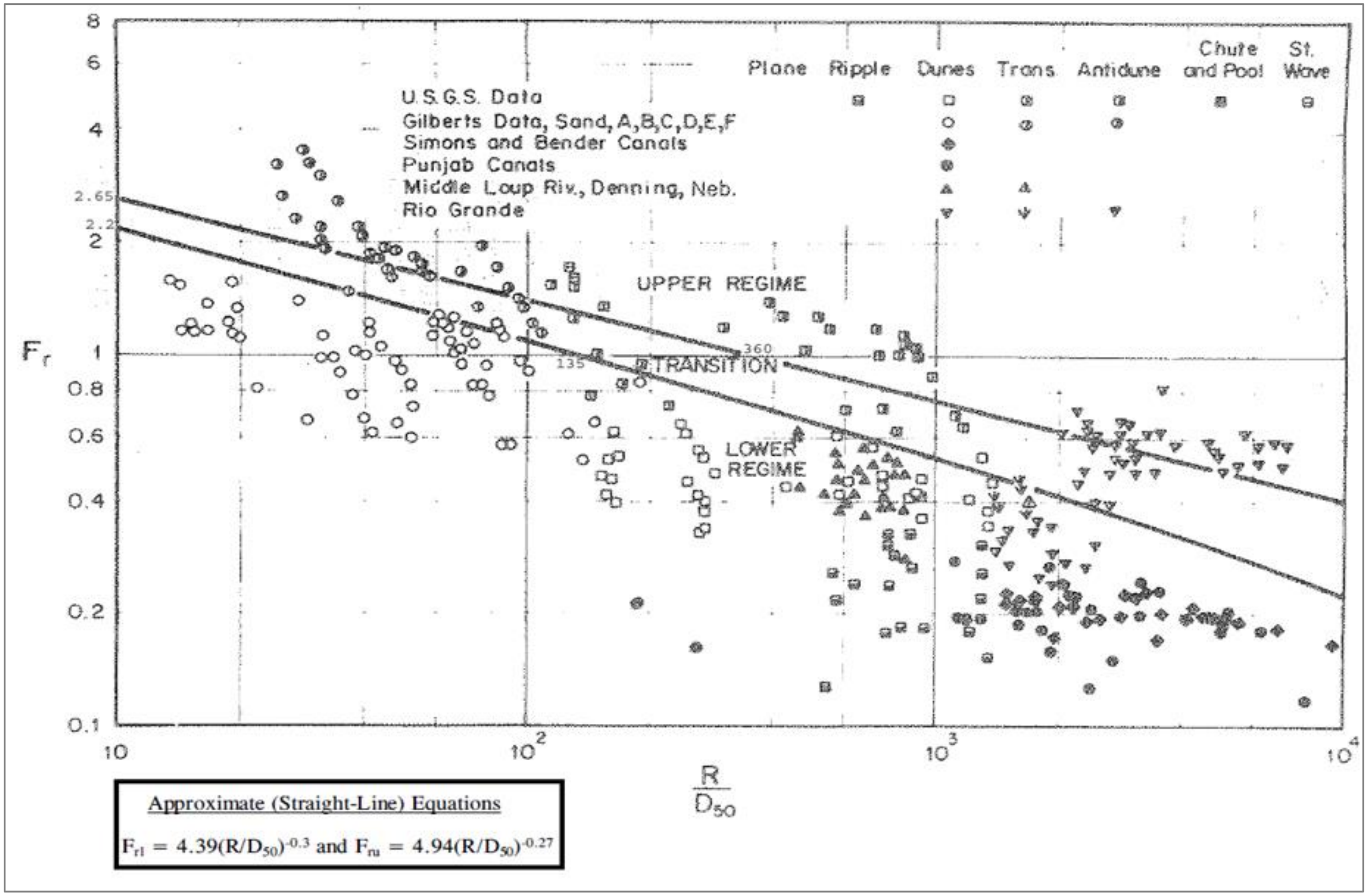

Figure 5.5 **Chart of Bed Form Flow Regimes**

5.16 It is customary to consider the bed form scour component in upper regime flow as one-half of the anti-dune height, from crest to trough. Based on this relationship, an equation was developed (Simons, Li & Associates 1982). This relationship is:

$$Z_a = \frac{1}{2}(0.14)\frac{2\pi V_m^{\ 2}}{g} = 0.0137 V_m^{\ 2}\text{, Upper Regime Flow}$$

Where:

g	=	gravitational acceleration, in ft/s^2
V_m	=	Median velocity, in ft/s
Z_a	=	Anti-dune scour, feet

5.17 Similarly for lower regime flow, one-half of the dune height, from crest to trough, is typically used as the bed form scour component. Again this relationship is visibly present in the equation below (developed by Julien & Klassen 1995):

$$Z_d = \frac{1}{2}(2.5Y)\left(\frac{D_{50}}{Y}\right)^{0.3} = 1.25Y^{0.7}D_{50}^{\ 0.3}\text{, Lower Regime Flow}$$

Where:

Y	=	Flow depth, in feet
D_{50}	=	Median particle diameter, in feet
Z_d	=	Dune scour, feet

D_{50} is determined from a separate technical memorandum (Tetra Tech, 2015)).

5.18 For this reach, it was determined from the channel hydraulics that, for the purposes of this investigation, the entire reach can be considered to be in lower regime flow conditions.

Low-Flow Channel Incisement

5.19 Low-flow channel incisement is the formation of a low-flow channel within the main channel in which low discharges are carried. There is no known methodology for predicting low-flow channel depth. Based on guidance as presented by Zeller (1981), if a low-flow thalweg is predicted to be present, it should be assumed to be at least two feet deep within regional watercourses, unless field observations indicate otherwise. For Berryessa Alt 2A reach this value was assumed to be one foot since it is assumed that a low flow has already developed and an additional foot could develop once the Alternative 2A improvements are constructed in unprotected portions of the Alternative 2A reach. Low-flow channel incisement depth of 1 foot was assumed and used and used in the scour analysis.

Local Scour

5.20 Local scour is observed whenever an abrupt change in the direction of flow occurs. Abrupt changes in flow direction can be caused by obstructions to flow, such as bridge piers or abrupt constrictions at bridge abutments, and drop structures. For this case, seven bridges are located within the Alternative 2A study reach. Local scour due to the presence of bridge piers was considered for this analysis for each of the seven bridges between I-680 and Calaveras Boulevard. Based on the Sediment transport modeling results (see separate technical memorandum (Tetra Tech, 2015)) show little accumulation during the storm events and some locations have local scour. The sensitivity analysis as part of the sediment transport analysis (by

SCVWD) further shows that the upstream section limits the debris loading to this portion of the channel. Therefore, no debris analysis was included in the bridge piers.

5.21 The depth of scour at bridge piers is highly dependent upon the shape of the pier. A square-nosed pier causes the deepest scour and is computed from (Richardson et. al. 1975):

$$Z_{lsp} = 2.2Y\left(\frac{b_p}{Y}\right)^{0.65} F_u^{0.43}$$

Where:

Z_{lsp}	=	Local scour depth due to pier, in feet
Y	=	Flow depth, in feet
B_p	=	Pier width normal to flow direction; in feet
F_u	=	Upstream Froude number

5.22 Because abrupt changes in channel geometry were eliminated, local scour was only computed at the bridge pier sections and was not determined for reaches where channel improvements are proposed as a part of Alternative 2A.

Bend Scour

5.23 Bend scour normally occurs along the outside of bends and is caused by spiral, transverse currents, which form within the flow as the water moves through the bend. Presently, there is no single procedure that will consistently and accurately predict bend scour over a wide range of hydraulic conditions. However, a relationship was developed by Zeller (1981) for estimating bend scour in sand-bed channels based upon the assumption of the maintenance of constant stream power within the channel bend. This relationship is as follows:

$$Z_{bs} = \frac{0.0685 Y_{max} V_m^{0.8}}{Y_h^{0.4} S_e^{0.3}}\left[2.1\left(\frac{\sin^2(\alpha/2)}{\cos\alpha}\right)^{0.2} - 1\right]$$

Where:

Z_{bs}	=	Bend-scour component of total scour depth, in feet
V_m	=	Maximum velocity of flow immediately upstream of bend, in feet per second
Y_{max}	=	Maximum depth of flow immediately upstream of bend, in feet
Y_h	=	Maximum Hydraulic depth of flow immediately upstream of bend, in feet
S_e	=	Maximum Energy slope immediately upstream of bend (or bed slope for uniform-flow conditions), in feet per foot
α	=	Angle formed by the projection of the channel centerline from the point of curvature to a point which meets a line tangent to the outer bank of the channel, in degrees.

5.24 The bend scour should be assumed to be zero for bends with deflection angles up to 17.8-degrees or if the radius of curvature divided by the channel top-width is greater than 10.

Total Scour

5.25 Total scour may be computed as the sum of general scour, bed form (anti-dune trough) depth, low-flow incision, local scour, and bend scour. An additional factor of safety of 1.2 is applied to account for the potential variability resulting from a non-uniform flow distribution. Table 5.3 below shows the maximum total scour as computed for the Alternative 2A reach.

Table 5.3 Total Single Event Scour Summary

Typical Section	Berryessa Creek River Sta. (-)	General Scour Depth Zgs (ft)	Maximum Anti-Dune Depth Za or Zd (ft)	Low-Flow Thalweg Depth Zlft (ft)	Bend Scour Depth Zbs (ft)	Sum of Scour Components	Total Scour Depth Zt=1.2x(∑Zi) Zt (ft)
1	87+20 to 89+10	1.00	2.29	1.00	0.00	4.29	5.15
1	89+10 to 92+21	1.00	2.24	1.00	0.00	4.24	5.09
2	93+76 to 114+73	1.00	2.20	1.00	0.00	4.20	5.04
3	115+23 to 124+53	1.00	1.71	1.00	0.00	3.71	4.45
4	125+91 to 164+43	1.00	2.05	1.00	0.00	4.05	4.86
5	167+56 to 174+90	1.00	1.72	1.00	0.00	3.72	4.46
5	174+90 to 178+23	1.00	1.52	1.00	1.24	4.76	5.71
5	178+23 to 191+00	1.00	1.51	1.00	0.00	3.51	4.21

5.26 The total computed maximum potential scour was determined as approximately 4.21 to 5.71feet throughout the Berryessa Creek Alternative 2A reach. Due to the potential channel scour, rip rap channel bottom and/or toe protections are provided throughout Alternative 2A reach. As previously mentioned, local scour computed based on the hydraulic influence of bridge piers was computed at each bridge structure within the reach and will be considered within the immediate vicinity of the bridge piers themselves, as applicable.

5.27 Based upon the bridge scour results in Table 5.4, the proposed invert and channel side slopes are proposed to be fully lined with either concrete or rip rap protection.

Table 5.4 Total Single Event Scour Summary

River Station	Bridge	General Scour Depth Zgs (ft)	Maximum Anti-Dune Depth Za or Zd (ft)	Low-Flow Thalweg Depth Zlft (ft)	Local Pier Scour Depth Zlsp (ft)	Sum of Scour Components	Total Scour Depth Zt=1.2x(∑Zi) Zt (ft)
161+00	New UPRR	1.00	1.89	1.00	3.25	7.14	8.57
142+00	UPRR - Culvert	1.00	2.05	1.00	2.88	6.93	8.32
137+25	Ames	1.00	2.00	1.00	3.81	7.81	9.37
124+00	Yosemite	1.00	1.86	1.00	3.73	7.59	9.11
93+00	Los Coches	1.00	2.22	1.00	2.92	7.14	8.57
86+25	Calaveras	1.00	2.21	1.00	3.62	7.83	9.40

Revetment Rock Sizing Using CHANLPRO

5.28 Similar to the scour analysis, five representative HEC-RAS segments defined by the five typical sections in the 90 percent design were selected for rock revetment size analysis. HEC-RAS average channel hydraulics were computed and utilized to determine the required rock revetment size. From the HEC-RAS output, the maximum average flow velocities and the corresponding flow depth for each segment was selected as a conservative value for the revetment rock sizing. The Corps' Channel Protection Design (CHANLPRO) computer program was used to determine the required revetment rock size as summarized in Table 5.5.

Table 5.5 Computed Revetment Rock Gradations

ALT2A		100-YR										
Typ. Sec	From Station to Station	Max Velocity (ft/sec)	Shear Stress (lbs/ft^2)	Section Depth (ft)	Bend (Y/N)	Bend Radius (ft)	Top Width (ft)	D_{30} (Min) (in)	D_{90} (Min) (in)	D_{100} (Max) (in)	D_{50} (Max) (in)	D_{85}/D_{15}
1	87+20 to 89+10	6.89	0.63	10.18	YES	1000	80.75	4.4	6.4	9	6	1.7
1	89+10 to 92+21	7.02	0.66	9.85	N	N/A	N/A	4.4	6.4	9	6	1.7
2	93+76 to 114+73	7.95	1.11	9.73	N	N/A	N/A	4.4	6.4	9	6	1.7
3	115+43 to 123+00	9.81	1.49	6.65	N	N/A	N/A	7.3	10.6	15	10	1.7
Transition	123+00 to 125+91	16.80	4.77	6.78	N	N/A	N/A	26.3	38.0	54[1]	36	1.7
4	125+91 to 164+43	9.51	1.16	8.46	N	N/A	N/A	7.3	10.6	15	10	1.7
5	167+56 to 174+90	8.55	1.12	6.64	N	N/A	N/A	5.8	8.4	12	8	1.7
5	174+90 to 178+23	8.76	1.23	6.31	YES	185	41.25	7.3	10.6	15	10	1.7
5	178+23 to 191+00	8.99	1.30	6.21	N	N/A	N/A	7.3	10.6	15	10	1.7

1. Grouted riprap with 24-inch thickness is used

Shear Stress Analysis

5.29 In the 60 percent design, the permissible shear and velocity criteria of Table 2 in *Stability Thresholds for Stream Restoration Materials* (Fischenich 2011) were adopted. The maximum shear stress, with adjustment for the spatial distribution, is computed based on Equations 9 and 10 presented in the Fischenich's paper as follow:

$$\tau_{max} = 1.5\gamma D S_f \; ; \; for\ straight\ channels$$

$$\tau_{max} = 2.65\,\gamma D S_f \left(\frac{R_c}{W}\right)^{-0.5} \; ; \; for\ sinuous\ channels$$

where γ is the specific weight of water, D is the hydraulic radius, S_f is the friction slope, R_c is the radius of curvature, and W is the top width of the channel. Additional factor of 1.15 is applied to account for the temporal maximums in turbulent flows. Detailed shear stress computations are presented in Appendix B. This data was used in in determining the requirements for channel side slope protection, types of protection to be utilized and limits.

5.30 Utilizing the HEC-RAS velocity distributions (Figure 5.6) option, channel shear stress (γDS_f) was computed along the channel wetted perimeter for the maximum "n" value analysis. Next the maximum shear stress is computed based on the equations stated in section 5.28 for either straight or curve channel reach. Table 5.6 shows the velocity distribution, shear stress, and maximum shear stress computations at STA 114+73.

Table 5.6 Computations of Maximum Shear Stress at STA 114+73

Left Station (ft)	Right Station (ft)	Flow (cfs)	Area (sq ft)	W.P.[1] (ft)	Percent Conv[2]	Hydr[3] Depth (ft)	Velocity (ft/s)	Shear (lb/sq ft)	τ_{max} (lb/sq ft)
20.02	24.30	0.00	0.00	0.07	0.00	0.02	0.12	0.00	0.00
24.30	28.59	9.19	4.73	4.79	0.30	1.10	1.94	0.16	0.28
28.59	32.87	55.46	13.90	4.79	1.78	3.25	3.99	0.48	0.83
32.87	37.15	129.05	23.07	4.79	4.15	5.39	5.59	0.80	1.38
37.15	41.43	224.82	31.72	4.62	7.22	7.41	7.09	1.13	1.95
41.43	45.72	262.20	33.75	4.28	8.43	7.88	7.77	1.30	2.24
45.72	50.00	262.20	33.75	4.28	8.43	7.88	7.77	1.30	2.24
50.00	54.28	262.20	33.75	4.28	8.43	7.88	7.77	1.30	2.24
54.28	58.56	262.20	33.75	4.28	8.43	7.88	7.77	1.30	2.24
58.56	62.85	262.20	33.75	4.28	8.43	7.88	7.77	1.30	2.24
62.85	67.13	262.20	33.75	4.28	8.43	7.88	7.77	1.30	2.24
67.13	71.41	262.20	33.75	4.28	8.43	7.88	7.77	1.30	2.24
71.41	75.69	262.20	33.75	4.28	8.43	7.88	7.77	1.30	2.24
75.69	79.98	262.20	33.75	4.28	8.43	7.88	7.77	1.30	2.24
79.98	84.26	191.47	29.22	4.79	6.15	6.82	6.55	1.01	1.74
84.26	88.54	102.14	20.05	4.79	3.28	4.68	5.09	0.69	1.19
88.54	92.82	36.85	10.87	4.79	1.18	2.54	3.39	0.37	0.64
92.82	97.11	3.19	2.15	3.28	0.10	0.73	1.48	0.11	0.19

1. Wetted perimeter.
2. Percent of conveyance.
3. Hydraulic depth.

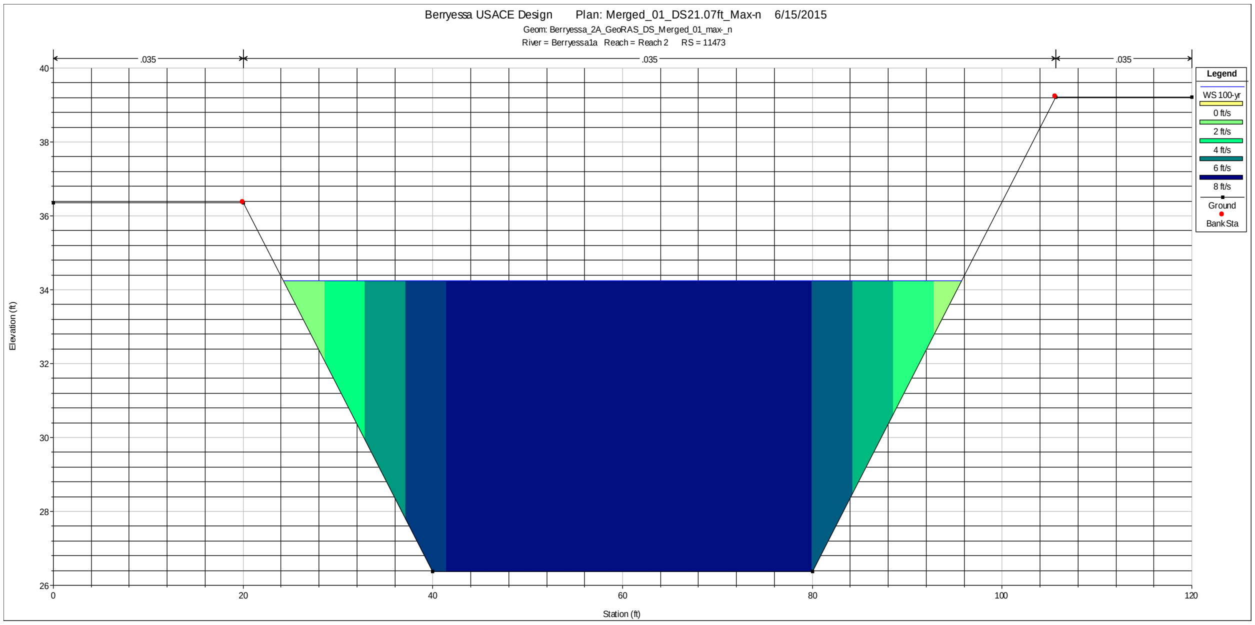

Figure 5.6 Shear Distribution HEC-RAS Cross Section 114+73

5.31 Soil conditions range from firm clays to sandy loam. Between Ames and Montague, the soils are in the range of sandy loam to firm loam with the remaining bank soils on the project classified as firm clays. Permissible shear stress for these soils range from 0.03 – 0.26 (lbs/sq ft). Vegetation coverage through the project varies but can be generally characterized as short native and bunch grass with average to good coverage and a permissible shear stress of 0.7 – 0.95 (lbs/sq ft). See pictures below for typical coverage through the channel limits.

Figure 5.7 Typical Channel Vegetation Coverage

5.32 Acceptable erosion control should possess a safety factor in excess of 1.2 to 1.3 based upon equation 11 presented in the Fischenich's paper as follow:

$$\mathrm{FS} = \frac{\tau_{\max}}{\tau est}$$

5.33 Utilizing a safety factor of 1.2 and assuming good vegetative coverage from short native and bunch grass, the upper acceptable limit for permissible shear stresses in the channel should be limited to 0.58 lbs/sq ft. Shear stresses above this limit will be susceptible to erosion and maintenance issues and will require additional erosion protection measures.

5.34 In locations where good vegetative coverage cannot be guaranteed, the acceptable limit for permissible shear stresses should be limited to the bare soil values in Table 2 presented in the Fischenich's paper.

5.35 Shear Stress values presented in Appendix B show the need for erosion protection for large portions of the channel embankment and invert. In line with the selected plan, erosion protection is provided with the combination of rip rap and native grasses. The erosion protection design consist of two sections of buried rip rap, a lower larger section primarily provides toe protection (for shear stresses above 2 lbs/ft^2) and a smaller rip rap rock section (for shear stresses between 0.58 to 2 lbs/ft^2), which extends up the channel embankment until the third section, consists of native natural grasses (for shear stresses below 0.58 lbs/ft^2), are able to withstand the shear stresses at the upper portions of the banks. Temporary erosion protection will be provided during the first approximately three years after construction through the use of a bio-degradable erosion protection that will help to increase the erosion resistance during the establishment period of hydro-seeded native grasses.

RISK-BASED PROJECT PERFORMANCE

5.36 Project performance for the Berryessa Creek Flood Control Project Post Authorization Study was estimated using the Corps' risk-based Monte Carlo simulation program HEC-FDA (Flood Damage Analysis), Version 1.2.5a (USACE 2010). The HEC-FDA program integrates hydrology, hydraulics, geotechnical, and economic relationships to determine damages, flooding risk, and project performance. Uncertainty is incorporated for each relationship and the model samples from a distribution for each observation to estimate damage and flood risk. The Berryessa Creek model includes the following relationships for each economic impact area:

- Discharge-Probability (with uncertainty determined by period of record)
- Stage-Discharge (stage in the channel with estimated error in feet)
- Stage-Damage (not used in this application, dummy values added to run program)

5.37 The selected plan developed for this study focused on the Federal Emergency Management Agency (FEMA) certifiable standards as defined in USACE EC 1110-2-6067. The EC lays out the criteria for determining acceptable top of levee/channel elevations in terms of risk-based project performance.

Methodology

Analysis Criteria

5.38 Risk and uncertainty principles were used in developing the selected plan to ensure that the design provides the best benefit to cost ratio. It should be further noted that for this project, the selected plan is to provide flood protection with floodwalls with a CNP of 0.76 for the 1 percent chance exceedance flood event. This plan is based upon a cost benefit analysis.

5.39 The criteria presented in the USACE EC 1110-2-6067 *Certification of Levee Systems for the National Flood Insurance Program*, dated August 31, 2010 for certification of a riverine levee system are as follows:

- The conditional non-exceedance probability (CNP) must be greater than 90 percent from overtopping of the 1 percent chance exceedance flood event for all reaches of the levee system.
- If the top of levee elevation is less than 3 feet above the FEMA base flood elevation, the levee can only be certified if the CNP is greater than 95 percent.
- The top of levee elevation shall not be less than 2 feet above the FEMA base flood elevation in any event, regardless if the CNP is 95 percent or greater.
- For incised channels, the top of channel elevation should be checked for containment of the 90 percent assurance flood level; containment of the 1 percent annual chance exceedance flood; and in accordance with the “freeboard” guidance provided in EM 1110-2-1601, *Hydraulic Design of Flood Control Channels*.

5.40 It is important to note that this assurance is only for hydrologic/hydraulic containment flood events by levees or incised channels; it does not include the probability of failure by any other mode (e.g., geotechnical) or the combined probability of all failure modes.

Analysis Method

5.41 Risk-based project performance was used to ensure that the designs meet the FEMA certification criteria presented in the previous section. To accomplish this, HEC-FDA version 1.2.5a was used to determine the CNP for the selected plan. This section describes the methodologies followed to determine the top of levee elevations and to analyze entrenched channel reaches.

Inputs

5.42 In developing a risk-based project performance model, a number of different inputs are required. The following inputs were developed for the Berryessa Creek analysis:

- Reaches and index point locations
- Hydrologic
- Hydraulic
- Economic
- Top of levee elevation/ channel elevation

5.43 The following section describes each of the inputs used for the risk-based performance.

Reaches and Index Points

5.44 Reaches were developed by grouping similar sections of channel into one reach. The computed channel hydraulics (e.g., flow depth, flow velocity, etc.) are fairly uniform due to the proposed uniform channel section within each reach. Therefore, one or more representative cross sections are chosen for each reach as the index point. This index point is the location where the hydraulic, hydrologic, and economic inputs are assigned for that reach. The chosen index points are selected based upon the cross section within in each reach with the least hydraulic capacity. The reach description and index points are listed in Table 5.7.

Table 5.7 Reach Description for Study Area

Index Point	Reach	Approximate Station
1	Los Coches Street to Calaveras Boulevard	88+67.89
2	Los Coches Street to Calaveras Boulevard	90+20.00
3	Los Coches Street to Calaveras Boulevard	91+30.00
4	Yosemite Drive to Los Coches Street	93+25.00
5	Yosemite Drive to Los Coches Street	100+00.00
6	Yosemite Drive to Los Coches Street	102+80.00
7	Yosemite Drive to Los Coches Street	103+50.00
8	Yosemite Drive to Los Coches Street	116+00.00
9	Yosemite Drive to Los Coches Street	119+80.00
10	Ames Avenue to Yosemite Drive	131+30.00
11	UPRR Culvert to Ames Avenue	140+00.00
12	UPRR Triple Box to UPRR Culvert	145+00.00
13	UPRR Triple Box to UPRR Culvert	152+98.90
14	Montague Expressway to UPRR Trestle	168+91.80
15	I-680 to Montague Expressway	171+50.00
16	I-680 to Montague Expressway	176+50.00
17	I-680 to Montague Expressway	181+00.00
18	I-680 to Montague Expressway	188+00.00
19	I-680 to Montague Expressway	195+80.00

Hydrologic Inputs

5.45 HEC-FDA allows for the entry of eight standard percent-chance exceedance events. The events used were the 50, 20, 10, 4, 2, 1, 0.4, and 0.2 percent-chance exceedance events. Inflow hydrographs (NHC 2003) for the Berryessa Creek were taken directly from the GRR model unsteady output for the 2-year through the 500-year events. Key points were identified where flow changes occurred and were converted to steady state analysis for the advancement of the Alternative 2A design. The discharge-probability values used for each index point are presented in Table 5.7 and Tables 5.8 and 5.9 list the hydrologic curve (discharge-probability curve) assigned to each reach.

Table 5.8 HEC-FDA Hydrologic Curves Input

Percent Chance Exceedance	Hydrologic Curve					
	1	**2**	**3**	**4**	**5**	**6**
50%	490	620	660	878	1,130	1,130
20%	700	947	1,009	1,379	1,737	1,737
10%	954	1,224	1,294	1,733	2,173	2,173
4%	1,145	1,545	1,665	2,360	2,850	2,850
2%	1,403	1,730	1,870	2,680	3,365	3,365
1%	1,545	2,010	2,170	3,112	3,885	4,100
0.4%	1,611	2,445	2,633	3,485	4,355	4,750
0.2%	1,770	2,693	2,970	3,800	4,640	5,037

Table 5.9 Reach Hydrologic Curve Assignment

Hydrologic Curve	Reach & Index Points
1	I-680 to UPRR Culvert– Index Points #12, #13, #14, #15, #16, #17, #18, and #19
2	UPRR Culvert to D/S of Ames Avenue – Index points #10 and #11
3	D/S of Yosemite Drive to D/S of Piedmont Creek - Index points #8 and #9
4	D/S of Piedmont Creek to D/S of Los Coches Creek - Index point #4, #5, #6, and #7
5	D/S of Los Coches Creek to U/S of Calaveras Blvd. - Index point #2 and #3
6	U/S of Calaveras Blvd – Index point #1

5.46 Confidence limits were applied to the hydrologic data using the guidelines presented in EM 1110-2-1619, *Engineering and Design Risk-based Analysis for Flood Damage Reduction Studies*, dated August 1996. An equivalent period of record of 35 years used in the development of alternatives and the approved GRR (Tetra Tech 2012) was adopted and applied to the hydrologic data for all reaches and was used by the HEC-FDA program to calculate the confidence limits.

Hydraulic Inputs

5.47 The hydraulic data inputs for each reach were taken from the HEC-RAS modeling of the selected plan for this study. The 50, 20, 10, 4, 2, 1, 0.4, and 0.2 percent-chance exceedance event stage data were imported into the HEC-FDA model for each index location. An error in the water surface stage was applied to the hydraulic data using the guidelines presented in EM 1110-2-1619 *Engineering and Design Risk-based Analysis for Flood Damage Reduction Studies*, dated August 1996. The stage error was computed by HEC-FDA using the standard deviation of the error range. The standard deviation was developed using the results from HEC-RAS model runs using high and low Manning's n values for the selected plan. It should be noted that the Manning's n value of 0.030 (Table 5.10) used in the proposed channel model was considered as the normal value, and the assumed high and low Manning's n values were 0.027 and 0.035 per Table 5.5 of *Open Channel Hydraulics* (Chow 1959), respectively.

Table 5.10 Computed 100-Year Water Surface Elevations

Variable	Channel Conditions	Recommended Value	Normal	Minimum	Maximum
Basic, n_b					
	Earth	0.020			
	Rock cut	0.025			
	Fine gravel	0.024	0.024[1]	0.024	0.024
	Coarse gravel	0.028			
Degree of Irregularity, n_1					
	Smooth	0.000	0.000[2]	0.000	0.000
	Minor	0.005			
	Moderate	0.010			
	Severe	0.020			
Variation of Channel Cross Section, n_2					
	Gradual	0.000	0.000[3]	0.000	0.000
	Alternating occasionally	0.005			
	Alternating frequently	0.010 - 0.015			
Relative Effect of Obstructions, n_3					
	Negligible	0.000	0.000[3]	0.000	0.000
	Minor	0.010 - 0.015			
	Appreciable	0.020 - 0.030			
	Severe	0.040 - 0.060			
Vegetation, n_4					
	Low	0.005 - 0.010	0.006[4]	0.003[5]	
	Medium	0.010 - 0.025			0.011[6]
	High	0.025 - 0.050			
	Very High	0.050 - 0.100			.
Degree of Meandering, m					
	Minor	1.00	1.000	1.000	1.000
	Appreciable	1.15			
	Severe	1.30			
$n = (n_b + n_1 + n_2 + n_3 + n_4) \cdot m$			0.030	0.027	0.035

1. Channel bottom based on sediment samples; no vegetation
2. Composite section (Rip rap) with smooth transitions and sideslope
3. Prismatic Channel Section
4. Assumed drought conditions and little vegetation for minimum
5. Assumed maintained grass for normal
6. Assumed unmaintained grass for maximum

The mean water surface profile was computed using normal Manning's n-value with downstream water surface elevation at 21.07 feet (SCVWD 2015) and 3-foot-wide and 3-foot-tall pier floating debris per GRR; the maximum water surface elevation profile was computed using the maximum Manning's n-value with downstream water surface elevation at 21.87 feet (SCVWD 2015) and 3 feet wide and 3 feet tall pier floating debris per GRR and sediment depositions at upstream of UPRR trestle, UPRR culvert, and Los Coches Street per sediment transport analysis (Appendix B); the minimum water surface elevation profile was computed using the minimum Manning's n-value with downstream water surface elevation at 21.07 feet (SCVWD 2015) and 3 feet wide and 3 feet tall pier floating debris and no sediment depositions. It should be noted for the minimum Manning's n-value hydraulic analysis, that the computed water surface elevations upstream of Calaveras Boulevard were unrealistic for the pressure and/or weir flow option selected for high flow method in the HEC-RAS bridge routine. Review of the model simulations showed that for the 2-yr to 200-yr events the upstream water depth was at or below 1.2 times the culvert height. At this depth the crossing is operating in an open flow condition and not pressure and/or weir (Chow 1959). Due to the channel configuration, smooth transitions, and the headwater depth to culvert ratio, the energy equation was selected for use in the high flow method in the HEC-RAS bridge routine. This selection provided a practical result consistent with hydraulic modeling principals and standard practice. For the 500-year simulation the headwater depth to culvert ratio is above 1.2, therefore pressure and/or weir flow option is utilized for this event only in the minimum Manning's n-value model.

The standard deviation was developed from the following equation:

$$S_{model} = E_{mean} / 4$$

Where

S_{model} = standard deviation of error range

E_{mean} = mean stage difference between high and low Manning's n HEC-RAS runs

5.48 In addition to the modeling uncertainty, uncertainty due to natural variation should be combined with the values from the modeling uncertainty per equation 5-6 of EM 1110-2-1619 as follows:

$$S_t = \sqrt{S_{natural}^2 + S_{model}^2}$$

Where S_t is the standard deviation of the total uncertainty, $S_{natural}$ is natural uncertainty, and S_{model} is modeling uncertainty. The largest natural uncertainty of 0.41 feet used in the development of alternatives (Tetra Tech 2012) was computed using Equation 5-5 of EM 1110-2-1619 as follows:

$$S_{natiral} = \left[0.07208 + 0.04936\, I_{bed} - 2.2626 \cdot 10^{-7} + 0.02164 H_{Range} + 1.4194 \cdot 10^{-5} Q_{100}\right]^2$$

Where H_{range} is the maximum expected or observed stage range, A_{Basin} is the basin area in square kilometers, Q_{100} is the 100-year estimated discharges in cubic meters, and I_{bed} is a stream bed identifier. Details computation of $S_{natural}$ is provided in Appendix B. The value of 0.41 was adopted and used in this analysis.

5.49 Even though the E_{mean} values vary over the storm events but the S_{model} (=E_{mean}/4) values were limited by the minimum value of 0.3 feet. This makes the adopted S_{model} values fairly constant except some instance as Index Point 1 shown in Table 5.11. The computed 100-year water surface elevations of the 19 index points were summarized in Table 5.11 to illustrate the range of standard deviation of the total stage uncertainty. It should be noted that the total stage uncertainties associated with other return frequencies were coded into the HEC-FDA model.

Table 5.11 Computed 100-Year Water Surface Elevations

Index Point	Water Surface [1] (feet)	Water Surface [2] (feet)	E_{mean} (feet)	S_{model} [3,4] (feet)	S_{model} [5] (feet)	$S_{natural}$ (feet)	S_t [3] (feet)
1	29.48	28.06	1.42	0.36	0.36	0.41	0.54
2	29.75	28.37	1.38	0.35	0.35	0.41	0.54
3	29.89	28.51	1.38	0.35	0.35	0.41	0.54
4	30.62	29.29	1.33	0.33	0.33	0.41	0.53
5	31.33	29.92	1.41	0.35	0.35	0.41	0.54
6	31.72	30.34	1.38	0.35	0.35	0.41	0.54
7	31.83	30.46	1.37	0.34	0.34	0.41	0.53
8	35.18	34.24	0.94	0.23	0.30	0.41	0.51
9	37.01	36.08	0.93	0.23	0.30	0.41	0.51
10	44.71	43.93	0.78	0.20	0.30	0.41	0.51
11	48.69	47.89	0.80	0.20	0.30	0.41	0.51
12	52.24	51.15	1.09	0.27	0.30	0.41	0.51
13	54.26	53.25	1.01	0.25	0.30	0.41	0.51
14	60.41	59.68	0.73	0.18	0.30	0.41	0.51
15	61.28	60.44	0.84	0.21	0.30	0.41	0.51
16	63.49	62.65	0.84	0.21	0.30	0.41	0.51
17	65.68	64.85	0.83	0.21	0.30	0.41	0.51
18	69.10	68.26	0.84	0.21	0.30	0.41	0.51
19	73.31	72.54	0.77	0.19	0.30	0.41	0.51

1. Water surface elevations based on high Manning's n-values.
2. Water surface elevations based on low Manning's n-values.
3. Round to two decimal points.
4. Computed by $E_{mean}/4$.
5. Minimum standard deviation of error in stage per Table 5-2 of EM 1110-2-1619.

Economic Inputs

5.50 As the name suggests, HEC-FDA is primarily used as a flood damage analysis tool, of which project performance is one aspect. Therefore, economic inputs in the form of stage-damage curves and floodplain structure locations are required to execute the program, although no actual economic analysis was performed as a part of the current study. The economic inputs are independent of the project performance results. For analyses performed for this study, one dummy damage curve and one dummy structure were entered into the HEC-FDA model. This economic data consisted of one data point and was used only to allow the calculation of the CNP. It did not affect the performance evaluation or represent any particular structure in the floodplain.

Top of Channel Elevations

5.51 The top of channel elevations were used as the target for the HEC-FDA program to determine the CNP for each reach of the selected plan. A top of levee/channel elevation was entered for all reaches based on the 60 percent design and the analysis methodology for that reach.

Project Performance Results

5.52 The risk-based project performance was determined according to the methodologies described above for each Index Point using the HEC-FDA program.

5.53 The AEP and long-term risk under proposed top of channel elevations are shown in Table 5.12. Based on these results, there is 1.1 percent chance of flooding in a given year due to overtopping for the channel analyzed along the proposed improvement reach.

Table 5.12 Proposed Top of Channel Elevations Performance

Index Point	Top of Proposed Channel Elevation[1] (feet)	Annual Exceedance Probability		Long-Term Risk (Probability of Exceedance over Indicated Time Period)		
		Median	Expected	10 years	30 years	50 years
1	30.50	0.0030	0.0047	0.0459	0.1315	0.2094
2	30.75	0.0031	0.0056	0.0545	0.1549	0.2446
3	31.22	0.0001	0.0034	0.0337	0.0978	0.1577
4	31.60	0.0033	0.0064	0.0623	0.1756	0.2752
5	31.30[2]/33.10[3]	0.0063/0.0001	0.0110/0.0012	0.1048/0.0114	0.2825/0.0338	0.4250/0.0557
6	32.34	0.0033	0.0065	0.0633	0.1781	0.2788
7	32.43	0.0001	0.0000	0.0003	0.0008	0.0013
8	36.97	0.0001	0.0000	0.0003	0.0008	0.0013
9	38.65	0.0001	0.0001	0.0010	0.0031	0.0052
10	45.72	0.0001	0.0027	0.0267	0.0780	0.1266
11	52.50	0.0001	0.0000	0.0002	0.0005	0.0009
12	55.82	0.0001	0.0000	0.0002	0.0005	0.0009
13	55.56	0.0001	0.0001	0.0013	0.0040	0.0067
14	64.00	0.0001	0.0000	0.0002	0.0005	0.0009
15	64.00	0.0001	0.0000	0.0002	0.0005	0.0009
16	64.75	0.0001	0.0009	0.0094	0.0278	0.0460
17	67.00	0.0001	0.0000	0.0005	0.0016	0.0027
18	70.50	0.0001	0.0000	0.0004	0.0012	0.0020
19	76.93	0.0001	0.0000	0.0002	0.0005	0.0009

1. Lowest top of bank elevation used in R&U analysis unless otherwise stated.
2. Top of right bank elevation.
3. Top of left bank elevation.

5.54 Table 5.13 shows the CNP, which indicates the probability of the channel successfully containing a given flood frequency event without overtopping. The FDA program analysis found that CNPs for a 100-year flood event (i.e., a 0.01 exceedance probability event) at Index Points #1, #2, #3, #4, #5, #6, and #7 did not meet the minimum USACE/FEMA requirement of 90 percent. However, with a CNP of 0.76 for the 1 percent chance exceedance flood event, the flood protection objective of the selected plan can be met with the addition of floodwalls.

Table 5.13 Proposed Top of Channel Elevations Conditional Non-Exceedance Probability

Index Point	Approximate Station	Conditional Non-Exceedance Probability by Events					
		10%	4%	2%	1%	.4%	.2%
1	88+67.89	1.0000	0.9960	0.9523	0.8018	0.6464	0.6100
2	90+20.00	1.0000	0.9885	0.9097	0.7786	0.6481	0.6069
3	91+30.00	1.0000	0.9937	0.9455	0.8639	0.7801	0.7534
4	93+25.00	1.0000	0.9753	0.9032	0.7604	0.6196	0.5617
5	100+00.00[1]	1.0000	0.9421	0.8204	0.6158	0.4394	0.3693
5	100+00.00[2]	1.0000	0.9963	0.9839	0.9561	0.9270	0.9158
6	102+80.00	1.0000	0.9734	0.9006	0.7607	0.6219	0.5634
7	103+50.00	1.0000	0.9740	0.9015	0.7615	0.6244	0.5676
8	116+00.00	1.0000	1.0000	1.0000	0.9998	0.9992	0.9991
9	119+80.00	1.0000	1.0000	0.9994	0.9975	0.9928	0.9913
10	131+30.00	1.0000	0.9924	0.9696	0.9111	0.8077	0.7776
11	140+00.00	1.0000	1.0000	1.0000	0.9999	0.9997	0.9997
12	145+00.00	1.0000	1.0000	0.9997	0.9995	0.9995	0.9995
13	152+98.90	1.0000	0.9995	0.9968	0.9995	0.9945	0.9938
14	168+91.80	1.0000	1.0000	0.9998	0.9998	0.9998	0.9997
15	171+50.00	1.0000	1.0000	0.9998	0.9998	0.9998	0.9997
16	176+50.00	1.0000	0.9958	0.9784	0.9684	0.9617	0.9555
17	181+00.00	1.0000	0.9999	0.9990	0.9984	0.9981	0.9979
18	188+00.00	1.0000	1.0000	0.9996	0.9990	0.9989	0.9988
19	195+80.00	1.0000	1.0000	0.9998	0.9998	0.9997	0.9997

Note: red highlighted values are CNP values less than 90% for 1% AEP flood.
1. Top of right bank elevation.
2. Top of left bank elevation.

5.55 The computed 100-year water surface elevations and freeboard at each Index point are shown in Table 5.14.

Table 5.14 Proposed Top of Channel Elevations and Freeboard

Index Point	Approximate Station	100-year WSEL[1] (ft)	Top of Channel Elevation (ft)	Freeboard (ft)
1	88+67.89	28.61	30.50	1.89
2	90+20.00	28.90	30.75	1.85
3	91+30.00	29.04	31.22	2.18
4	93+25.00	29.77	31.60	1.83
5	100+00.00[2]	30.43	31.30[2]	0.87
5	100+00.00[3]	30.43	33.10[3]	2.67

Table 5.14 Proposed Top of Channel Elevations and Freeboard

Index Point	Approximate Station	100-year WSEL[1] (ft)	Top of Channel Elevation (ft)	Freeboard (ft)
6	102+80.00	30.85	32.34	1.49
7	103+50.00	30.96	32.43	1.47
8	116+00.00	34.42	36.97	2.55
9	119+80.00	36.39	38.65	2.26
10	131+30.00	44.24	45.72	1.48
11	140+00.00	48.80	52.50	3.70
12	145+00.00	51.36	55.82	4.46
13	152+98.90	53.59	55.56	1.97
14	168+91.80	60.00	64.00	4.00
15	171+50.00	60.80	64.00	3.20
16	176+50.00	63.50	64.75	1.25
17	181+00.00	65.18	67.00	1.82
18	188+00.00	68.59	70.50	1.91
19	195+80.00	72.83	76.93	4.10

1. Water surface elevation based on normal Manning's n-values including superelevation.
2. Top of right bank elevation.
3. Top of left bank elevation.

5.56 At Index Point 5, to provide a minimum CNP value of 0.76, the design channel bank should be 32.10 feet. In this area (Figure 5.8), approximately 200 linear feet (Sta. 99+00 to Sta. 101+00), there is a localized depression within the UPRR spruce line tracts. It would only allow for a maximum channel bank elevation of 31.30 feet without the utilizations of a floodwall or levee section. Based upon the non-damaging impacts within this localized area, the direction of topographic relief, added construction cost, and reduced long-term maintenance cost, it was determined that the reduced bank elevation to 31.30 is acceptable. It should maintain a minimum freeboard depth of 0.87 feet for the 0.01 exceedance storm event.

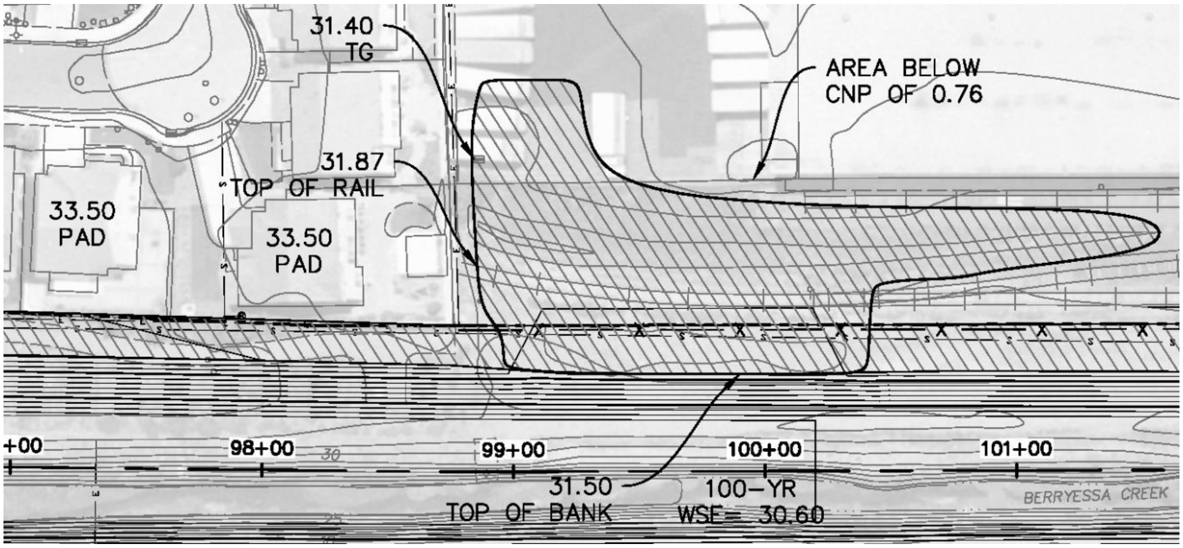

Figure 5.8 Localized Depression

SEDIMENT TRANSPORT

5.57 This section summarizes the results of the sediment transport analysis that was conducted to support the project design. The analysis includes sediment transport modeling of the baseline (existing) and with-project (proposed) conditions. Descriptions of the assumptions, inputs, methodologies, and detailed results of this analysis are provided in a separate technical memorandum (Tetra Tech, 2015).

5.58 Results from the baseline sediment transport model show the maximum degradation depth is generally less than 3 feet, with up to 4 feet of degradation predicted in the most entrenched areas downstream of Old Piedmont Bridge face (RS 23454) and upstream of drop structure at San Jose pump station (RS 17755). The largest amount of aggradation (up to 4 feet) is predicted near Sierra Creek confluence (RS 17092) and at the upstream face of I-680 Bridge and drop (RS 14042). Downstream from I-680 bridge, the channel is relatively stable with exception of a depositional reach between RS 10931 and 10226 (max 2.5 feet of aggradation) and a degradational reach downstream of RS 2881 (max 2 feet of degradation).

5.59 Results from the proposed conditions sediment-transport model show that the project reach downstream of I-680 bridge (downstream of RS 20143) is not experiencing significant bed change after the 100-year single event (the maximum bed change is 1.6 feet for the Yang function; the minimum bed change is -1.8 feet for the Yang function). A depositional zone is predicted at the new UPRR trestle bridge (RS 16175) with maximum 1 foot of deposition. Other depositional zones are found upstream of the pedestrian bridge at RS 16703 with maximum 0.3 feet of deposition; at the UPRR culvert (RS 14292) with maximum 1.5 feet of deposition; upstream of Los Coches Street (RS 9376) with maximum 1.5 feet of deposition; and upstream of Calaveras Boulevard (RS 8720) with maximum 0.2 feet of deposition for the 100-year design event. Upstream of the I-680 bridge, significant deposition (on the order of 6 feet) is predicted immediately downstream of the drop structure at RS 20542, which is a similar trend simulated in the baseline conditions. A potentially degradational reach with maximum 0.5 feet of scour is predicted downstream of the I-680 bridge (RS 20132 to 19200) if the bed is left unprotected. Other degradational reaches are predicted downstream of the bed protection at RS 11473 with maximum 0.4 feet of scour; downstream of Los Coches Street (RS 9220) with maximum 1.3 feet of scour; and downstream of Calaveras Boulevard (RS 8130) with maximum 1.8 feet of scour for the 100-year design event.

5.60 Although the sediment transport analysis show that the proposed condition is not experiencing significant bed change, analysis does have limitations and cannot be solely relied upon for long term channel performance. The channel will need to be inspected and monitored in accordance with the developed O&M manual and adaptive management plan to ensure that excessive scour or deposition does not occur and reduce channel capacity or stability.

5.61 A sensitivity analysis of sediment transport results was performed with respect to inflowing sediment load, transport function, and hydrologic input. The original sediment load in (labeled "Load" in the sensitivity tables below) was cut in half because it was found to overload the supply reach upstream of the Old Piedmont Bridge; the equilibrium load (transport function dependent) was also included. The alternative transport functions analyzed were the Meyer-Peter-Muller (MPM) gravel formulation and the Ackers-White (AW) transport function. Additional flood events considered were the 2-year hydrograph, as a representative of lower flows, and a recurring 10-year hydrograph, as an approximation of the channel-forming (dominant) discharge that substitutes a long-term natural hydrograph. For the channel-forming scenario, five 10-year hydrographs were stacked together to maximize the probability of its occurrence in the next 50 years [according to binomial distribution, five occurrences of the 10-year flow has the highest probability (18 percent) within the next 50 years].

5.62 The summary results of the sensitivity analysis (bed change at the end of the simulation period) for the project reach downstream of I-680 bridge are presented in sensitivity Tables 5-15 to 5-17.

Table 5.15 Maximum Bed Change (feet)

Flood Event	Yang Transport Function			MPM Transport Function			AW Transport Function		
	Load	Half Load	Equilib. Load	Load	Half Load	Equilib. Load	Load	Half Load	Equilib. Load
100-YR	1.6	1.5	1.9	1.7	1.7	1.7	1.0	1.0	2.0
2-YR	1.6	1.6	1.7	1.4	1.4	1.4	0.6	0.6	0.6
Recurring 10-YR	2.5	2.5	2.7	2.2	2.2	2.2	1.0	1.1	2.6

Table 5.16 Minimum Bed Change (feet)

Flood Event	Yang Transport Function			MPM Transport Function			AW Transport Function		
	Load	Half Load	Equilib. Load	Load	Half Load	Equilib. Load	Load	Half Load	Equilib. Load
100-YR	-1.8	-1.8	-1.7	-1.2	-1.2	-1.2	-0.9	-0.9	-0.9
2-YR	-0.9	-0.9	-0.9	-0.5	-0.5	-0.5	-0.8	-0.8	-0.8
Recurring 10-YR	-3.0	-3.0	-3.3	-2.2	-2.2	-2.2	-1.0	-1.0	-1.8

Table 5.17 Average Bed Change (feet)

Flood Event	Yang Transport Function			MPM Transport Function			AW Transport Function		
	Load	**Half Load**	**Equilib. Load**	**Load**	**Half Load**	**Equilib. Load**	**Load**	**Half Load**	**Equilib. Load**
100-YR	0.0	0.0	0.1	0.1	0.1	0.1	0.0	0.0	0.2
2-YR	0.1	0.1	0.1	0.1	0.1	0.1	0.0	0.0	0.1
Recurring 10-YR	0.0	0.0	0.0	0.1	0.1	0.1	0.0	0.0	0.2

5.63 As indicated in the summary tables above, the results are relatively insensitive to the upstream sediment load as the project reach is 2 miles downstream of the inflowing load location (which is upstream of Old Piedmont Bridge). The channel-forming discharge scenario (recurring 10-year hydrograph) shows the largest magnitude of deposition (2.7 feet for Yang function; 2.2 feet for MPM function; and 2.6 feet for AW transport function) and scour (3.3 feet for Yang function; 2.2 feet for MPM function; and 1.8 feet for AW transport function), which is generally within a foot of difference from the 100-year flood results. The maximum differences in bed change between the three transport functions are 1-2 feet, which is not significant given all the uncertainties inherent in sediment transport modeling.

5.64 A long term sediment transport analysis to determine O&M sediment removal requirements for the project was performed by SCVWD (memorandum in Appendix B). The analysis predicts in the project reach, there are some sections that exhibit erosive characteristics, which seem to be limited to transition zones between natural and hardscape. However, most of the channel is armored and there is little to no aggradation in the project reach. The results from the modeling predict no future sediment removal maintenance for the Upper Berryessa project reach.

CONSTRUCTION PHASING

5.65 The Upper Berryessa Creek Flood Management Project may require construction phasing to avoid increasing flood risks downstream, while adhering to the project schedule. Hydraulic analyses were performed to identify possible staging opportunities. The recommended sections for construction extend from 500’ upstream of Piedmont Creek confluence to just downstream the UPRR trestle bridge, and from downstream I-680 to 1,200’ upstream of Montague Expressway. The analysis performed by SCVWD is provided in Appendix B.

References

American Society of Civil Engineers (ASCE). 2006, Sedimentation Engineering: Theory, Measurements, Modeling, and Practice, Hardcover, 424 pages.

Chow, V. T. 1959, Open Channel Hydraulics, McGraw-Hill, New York.

Fischenich, C. 2011. Stability Thresholds for Stream Restoration Materials. ERDC TN-EMRRP-SR-29. USACE Reach and Development Center, Environmental Laboratory, Vicksburg, VA.

Gordon E. Grant. February 1977, Critical Flow Constrains Flow Hydraulics in Mobile-bed Streams: a New Hypothesis, Water Resources Research, Vol. 33, No. 2, Pages 349-358.

Julien, P. Y., and Klassen, G. J. (1995) Sand-dune geometry of large rivers during floods. J. Hydraulic. Eng., 121 (9), 657–663.

Northwest Hydraulic Consultants (NHC). 2003. Berryessa Creek Watershed Hydrology Report. April 2003.

Northwest Hydraulic Consultants (NHC). 2006. Berryessa Creek Watershed Hydrology Report Addendum 1. October 2006.

Santa Clara Valley Water District (SCVWD). 2015. Starting Water Surface Elevation for Lower Penitencia and Lower Berryessa Creek Projects. Milpitas, California.

Simons, Li and Associates, Inc. Sept. 30, 1983, Hydraulic, Erosion and Sedimentation Study of the Santa Clara River, Ventura, California.

Tetra Tech, Inc. 2012. Berryessa Creek Project, Santa Clara County, California. Appendix B Engineering and Design, Part 1: Hydrologic Analysis of Alternatives. January 2012.

Tetra Tech, Inc. 2015. Berryessa Creek Sediment Transport Technical Memorandum, May 2015.

U.S. Army Corps of Engineers (USACE). August 1996. Engineering Manual No. 1110-2-1619, Engineering and Design: Risk-Based Analysis for Flood Damage Reduction Studies, Washington, DC

USACE. 1998, User's Manual for CHANLPRO, PC Program for Channel Protection Design, Waterways Experiment Station, Vicksburg, MS.

USACE. August 2005, Hydrology and Hydraulics Policy Memorandum No. 4, Debris Loading on Bridges and Culverts, Los Angeles District, California.

USACE. March 2008, HEC-RAS River Analysis System. Hydrologic Engineering Center. Davis, California.

USACE. March 2014. Berryessa Creek Element Coyote and Berryessa Creeks: Final General Reevaluation Report/ Environmental Impact Statement. Prepared by Tetra Tech, Inc. March 2014.

USACE. Sacramento District. 1993. Coyote and Berryessa Creeks, California, General Design Memorandum, Volume I or II, November 1993.

Zeller, M. E., 1981, Scour Depth Formula for Estimation of Toe Protection Against General Scour, Pima County Department of Transportation and Flood Control District, Tucson, Arizona.

6. GEOTECHNICAL BASIS FOR DESIGN

INTRODUCTION

6.1 The geotechnical work consisted of subsurface explorations, laboratory testing, geotechnical evaluations of the channel improvements, floodwalls, and culverts, and geotechnical recommendations for the structural designs and evaluations. In addition, geotechnical-related construction recommendations are presented in this section. Much of the geotechnical work was documented in the Geotechnical Appendix Report for the Upper Berryessa Creek Flood Risk Management Project that was prepared for the Santa Clara Valley Water District.

6.2 The subsurface exploration was performed in phases. The initial phase consisted of a review of the available historical subsurface information. This review was used to prepare a work plan for the subsequent exploration phases that consisted of cone penetration test (CPT) borings then standard penetration test (SPT) borings. The locations of the historical borings, CPT borings, and SPT borings are shown on Figures 6.1, 6.2, and 6.3, respectively.

6.3 Details of the findings and all of the historic, CPT, and SPT boring logs from the subsurface explorations are presented in the Geotechnical Appendix Report.

SUBSURFACE CONDITIONS

Soil Conditions

6.4 The subsurface conditions encountered in the exploratory borings generally consisted of shallow fills soils (af) overlying alluvial soils. The alluvium encountered in the borings were divided into two basic groups, younger alluvial deposits (Qa) associated with basin and younger alluvial fan deposits and older alluvial deposits (Qoa) associated with older alluvial fan deposits of the Upper Pleistocene and Holocene. Field classification between older and younger geologic units was primarily based on color and consistency of the soils observed.

6.5 Uncontrolled fill was encountered in all of the SPT borings at the ground surface to depths of 2 to 7 feet overlying natural soils. The uncontrolled fill consisted of silty sand or clayey sand in eight of the borings but consisted of clay soils in two of the borings. No documentation or records are available for this existing fill.

6.6 The natural soils beneath the uncontrolled fill typically consisted of firm cohesive soils with interbedded layers of sand to the depths of the borings. The cohesive soils were somewhat variable, ranging from clayey silts (CL-ML) to silty clays (CL) to high-plasticity clays (CH) that generally became stiffer with depth. The interbedded sands were generally silty sands and clayey sands.

6.7 The sand content of the cohesive soils also varied along the alignment. Some of the higher plasticity clays had 10 to 20 percent sand content, while many of the silty clays had 35 to nearly 50 percent sand content. While the sand content in the silty clays was high, it is believed that there is sufficient fine contents in these deposits such that their behavior will be more cohesive in nature rather than granular.

6.8 Softer zones of clays were encountered in several of the borings, although these layers were not thick and did not appear to be continuous. Many of these layers were encountered near the bottom of the existing channel invert elevation.

6.9 However, boring SPT-16 encountered 4 feet of clayey sand fill at the ground surface overlying stiff clay to a depth of 12 feet. Below the stiff clay, 13 feet of soft to medium stiff clay was encountered to a depth of 25 feet, where stiff clays were encountered to the depth of the boring. The N-values for the SPT samples in the soft to medium stiff layer were 4, although one sample exhibited an N-value of 3.

Groundwater Conditions

6.10 Historical high groundwater at the site was mapped by CDMG at depths between 7 and 12 feet (Figure 4, CDMG, 2001). Groundwater was encountered in many of the historical borings within the project limits at depths varying from approximately 7 to 16 feet below existing grade. Further south along the alignment, near I-680, groundwater was encountered at a depth of 30 feet or more below existing grade.

6.11 In the 10 SPT borings drilled for the project, groundwater levels were encountered at depths of 8.8 to 17.2 feet, which is similar to the findings in the historic borings. This water was often contained in sand seams or other more permeable zones. However, in two of the borings (SPT-9 and SPT-14) no water was encountered in the borings at the completion of drilling. In boring SPT-18, a wet gravel layer was encountered at a depth of 13.0 feet that extended to the depth of the boring at 19.5 feet.

6.12 Construction work for the proposed channel improvements will require excavation within and beneath the existing channel bottom. It should be anticipated that this work will encounter groundwater.

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Figure 6.1. Historic Boring Locations

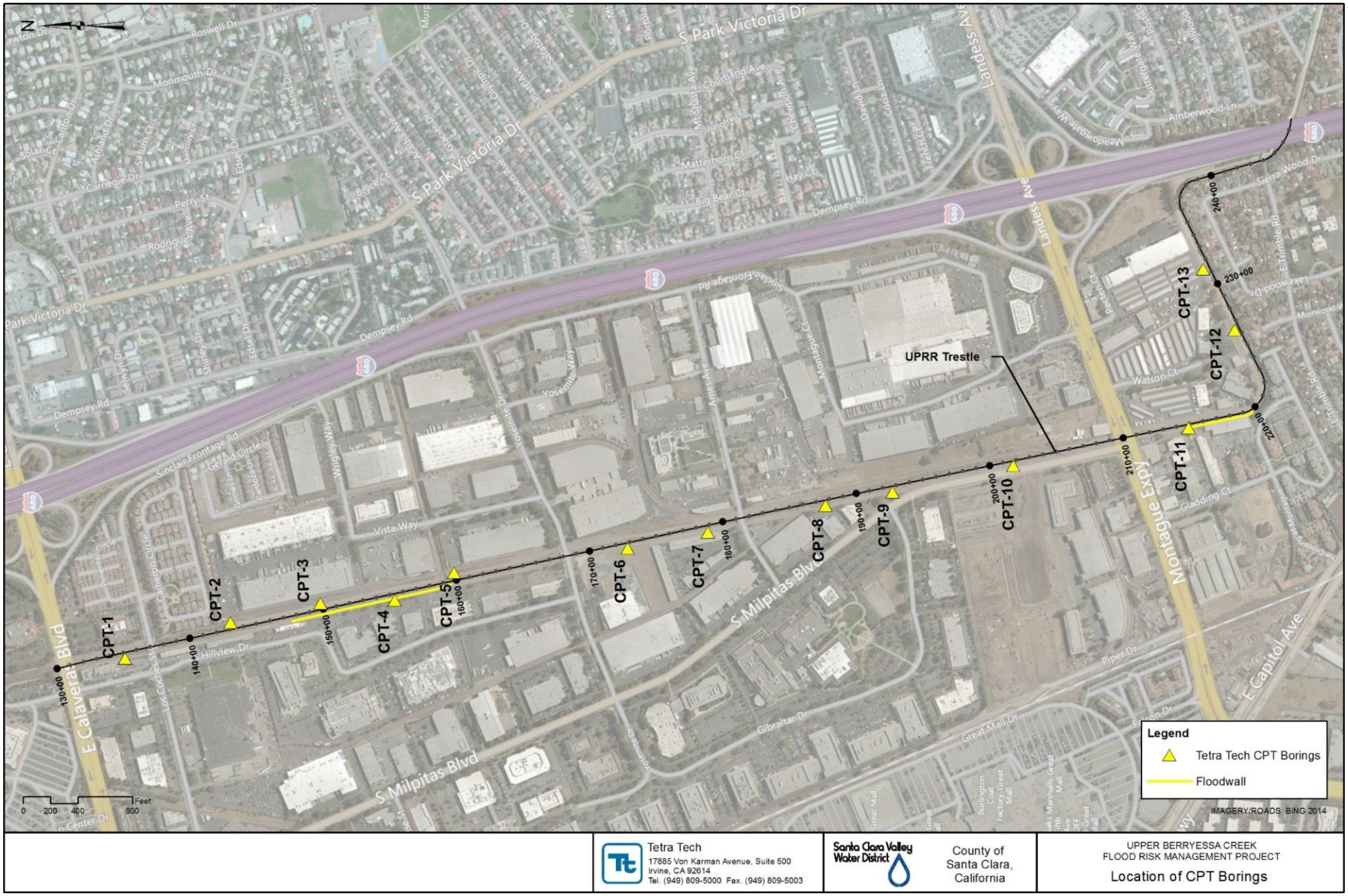

Figure 6.2 CPT Boring Locations

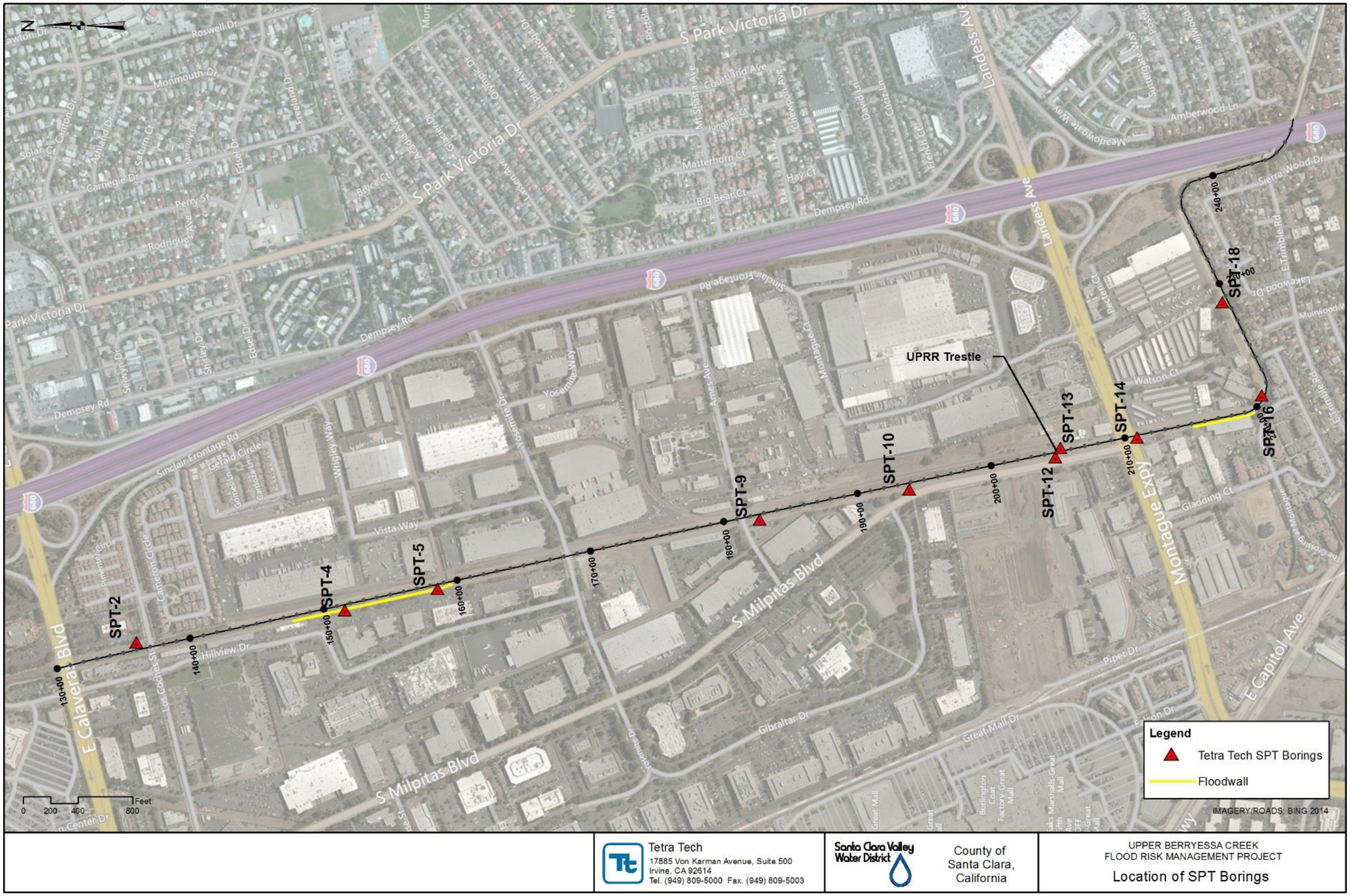

Figure 6.3 SPT Boring Locations

ENGINEERING SEISMOLOGY

General Seismic Setting

6.13 The Northern California region is known to be seismically active. Earthquakes occurring within approximately 60 miles of the site are generally capable of generating ground shaking of engineering significance to the proposed construction. The project area is located in the general proximity of several active and potentially active faults. The closest active faults to the site are the Hayward Fault, located approximately 1.1 mile to the northeast, and the Calaveras-Pacines-San Benito Fault (Hayward Fault), located approximately 4.2 miles to the east. The Calaveras and Hayward Fault splay apart south of the project site and become two distinct fault features. Other nearby faults include the Monte Vista/East Fault and San Andreas Fault, located approximately 11 miles and 15.5 miles to the southwest, respectively.

6.14 The most notable historic earthquakes occurred in 1906 (San Francisco earthquake) and 1989 (Loma Prieta earthquake).

Seismic Demand

6.15 The seismic demand at the site was evaluated based upon a probabilistic seismic hazard analyses approach. The evaluation utilized the United States Geological Survey (USGS) Probabilistic Seismic Hazard Deaggregation website https://geohazards.usgs.gov/deaggint/2008/ as a tool to calculate probabilistic peak ground acceleration. The attenuation relationships used for ground motion prediction include the Next Generation Attenuation (NGA) relationships of Boore and Atkinson (2008), Campbell and Bozorgnia (2008), and Chiou and Youngs (2008). An assumed average shear wave velocity in the top 30 meters (Vs_{30}) of 270 meters per second was used in the model. The peak ground accelerations for various year return periods were estimated from the USGS website. Corps' criteria for design of structures require various return period values for Operating Basis Earthquake (OBE) and Maximum Design Earthquake (MDE). A summary of the estimated peak ground acceleration values for various return periods are presented in **Table 6.1**.

Table 6.1 Estimated Peak Ground and Spectral Accelerations

Return Period	Peak Ground Acceleration	Spectral Acceleration		
		0.2 second	0.3 second	1 second
108 years	0.36g	0.77g	0.75g	0.43g
144 years	0.41g	0.87g	0.86g	0.50g
475 years	0.63g	1.35g	1.35g	0.82g
949 years	0.76g	1.64g	1.66g	1.04g

6.16 Seismic parameters for the Maximum Considered Earthquake (MCE) were estimated using the USGS website (**http://earthquake.usgs.gov/designmaps/us/application.php**). The MCE values estimated by this website are the lesser of values based on a probabilistic analysis utilizing a 2,475-year return period (2 percent probability of exceedance in 50 years) and maximum values based on a deterministic analysis of nearby characteristic faults. This procedure yielded design spectral acceleration values of 1.24g for 0.2 and 0.3 second, and 0.75g for 1.0 second. A printout of the MCE analysis is included in Appendix C.

Liquefaction Potential and Dynamic Settlement

6.17 Liquefaction of soils can be caused by ground shaking during earthquakes. Research and historical data indicate that loose, relatively clean granular soils are susceptible to liquefaction and dynamic settlement. Liquefaction is generally known to occur in saturated or near-saturated, cohesionless soils at depths shallower than about 50 feet. Most clayey silts, silty clays and clays are not typically adversely affected by ground shaking, however, fine-grained soils with high sensitivity (low remolded strength versus peak strength) can be susceptible to liquefaction.

Potential Liquefiable Soils

6.18 Evaluation of liquefaction potential for the sandy soils was performed based on the soil stratigraphy encountered in Boring SPT-12, and CPT sounding CPT-5, CPT-6, and CPT-8 through CPT-12. Potentially liquefiable soils consisted of relatively thin layers of loose to medium dense sandy soils encountered at various depths shown in the boring and CPT logs. In addition, fine-grained soils were evaluated with regard to strength sensitivity and susceptibility to liquefaction.

Groundwater Level

6.19 Historical high groundwater at the site was mapped by CDMG at depths of about 7 to 12 feet (CDMG, 2001). Parikh (2004) reported groundwater depths as shallow as 7.5 below the existing channel bank. For the current field exploration, groundwater shortly after the completion of drilling was encountered at depths of approximately 9 to 17 feet below the channel bank. In this study, a groundwater depth of 7 to 10 feet was assumed for evaluation of liquefaction potential of the on-site materials, depending on the boring/CPT location.

Evaluation of Liquefaction Potential

6.20 The liquefaction potential of cohesionless (sandy) soils was evaluated based on the field exploration and laboratory test results. Results of liquefaction analyses of granular soils are summarized in **Tables 6.2 and 6.3**. The analyses indicated that the loose to medium silty fine sands encountered at various depths are susceptible to liquefaction.

6.21 The plasticity index of the on-site clayey soils generally ranges from 15 to 52. Sensitivity analyses were performed for the on-site fine-grained soils with a plasticity index greater 18. Analyses of the sensitivity of the on-site clayey soils indicated low sensitivity with an estimated sensitivity index generally ranging from 1 to 4. Consequently, the potential for significant loss of strength of the on-site clayey soils and ensuing seismic deformation during seismic shaking is considered low. Results of sensitivity analyses for the on-site clayey soils are included in Appendix C.

Dynamic Settlement

6.22 Seismic settlement can occur in both dry and saturated sands when loose to medium-dense granular soils undergo volumetric changes during ground shaking. Seismic settlement can occur in saturated sands due to liquefaction or in dry sands due to densification of the soil matrix. **Tables 6.2 and 6.3** present the results of liquefaction analyses and dynamic settlement:

Table 6.2 Results of Liquefaction Analyses (108-year return period earthquake)

Boring No.	Assumed Groundwater Depth	Liquefiable Zone Depth	FS_{liq}	Liquefaction Settlement	Settlement of Dry Sands	Combined Dynamic Settlement
	(ft)	(ft)	–	(inch)	(inch)	(in)
SPT-12	10	14 to 16	0.9	0.5	0.1	0.6
CPT-5	7	Non - liquefiable	>1.3	--	--	--
CPT-6	7	Non - liquefiable	>1.3	--	--	--
CPT-8	10	Non - liquefiable	>1.3	--	--	--
CPT-9	10	Non - liquefiable	>1.3	--	--	--
CPT-10	10	Non - liquefiable	>1.3	--	--	--
CPT-11	10	Non - liquefiable	>1.3	--	--	--
CPT-12	10	Non - liquefiable	>1.3	--	--	--

Table 6.3 Results of Liquefaction Analyses (475-year return period earthquake)

Boring No.	Assumed Groundwater Depth	Liquefiable Zone Depth	FS_{liq}	Liquefaction Settlement	Settlement of Dry Sands	Combined Dynamic Settlement
	(ft)	(ft)	–	(inch)	(inch)	(in)
SPT-12	10	14 - 16	0.5	0.5	0.4	0.9
CPT-5	7	14 - 16	0.6 – 1.0	0.3	0.1	0.4
CPT-6	7	18 - 19	0.5 – 1.2	0.2	0.1	0.3
CPT-8	10	13 – 14, 27.5 - 29	0.5 – 1.0	0.6	0.1	0.7
CPT-9	10	10 - 11	0.5 – 0.6	0.5	0.1	0.6
CPT-10	10	10 - 14	0.9 – 1.3	0.4	0.0	0.4
CPT-11	10	17 – 20, 36 - 38	0.3 – 1.0	0.5	0.1	0.6
CPT-12	10	19 – 20.5	0.4 – 1.1	0.2	0.1	0.3

6.23 As shown in **Tables 6.2 and 6.3** above, the combined dynamic settlement was estimated to be less than 1 inch. Given the magnitude of the dynamic settlement and the thinness of the potentially liquefiable layers encountered in the exploration borings and CPTs, it is our opinion that liquefaction is not a geotechnical concern, and potential dynamic settlement at the site will not adversely impact the proposed improvements. The results of dynamic settlement analyses are presented in Appendix C.

ANALYSES OF CHANNEL IMPROVEMENTS

General

6.24 As mentioned previously, the channel improvements will be designed to provide protection against a 100-year level flood event. The improvements consist of regrading and widening the existing channel, installing slope protection on the channel slopes, and using short floodwalls less than 2 feet high in two areas. The following sections present the results of the analyses and evaluations for the proposed channel cross-sections.

Hydrologic and Hydraulic Evaluations

6.25 To determine the 100-year flood levels, the results of the hydrologic and hydraulic model were used. Refer to the discussion of the hydrologic and hydraulic model and results in Section 5 of this report for additional information.

Channel Geometry

6.26 The channel will be deepened slightly and the slopes will be graded to a consistent 2H: 1V slope and variable bottom width. Erosion protection will be placed on the channel slopes. It is anticipated that the erosion protection will consist of buried rip rap rock protection, biodegradable erosion control blanket and hydroseed with native grasses and wetland seed mix. Details of the erosion protection can be found in the design drawings. A typical cross-section of the proposed channel from the 60 percent design drawings is shown in **Figure 6.4.**

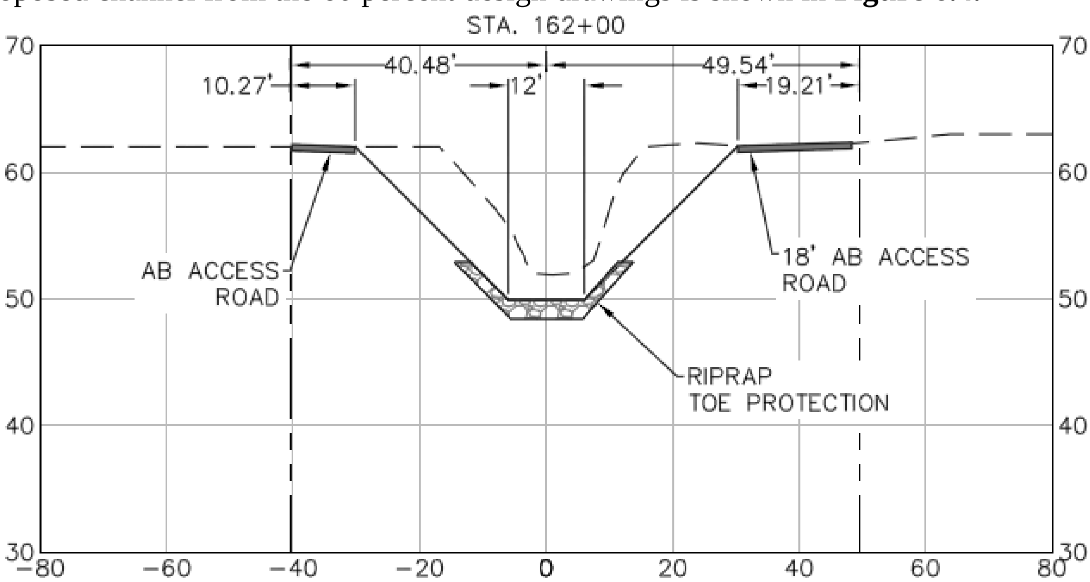

Figure 6.4 Typical Proposed Channel Cross-Section

Geotechnical Analyses

General

6.27 The geotechnical evaluations for the channel improvements consisted of slope stability analyses of the proposed side slopes using the results of the subsurface explorations and laboratory testing. The initial step in the evaluations was to review the results of the borings and laboratory testing and divide the project into reaches. A single cross-section was then analyzed for stability that would be representative for the entire reach. The most critical subsurface conditions encountered in the reach were used in the evaluations. Discussion of the reach determinations, shear strength determinations, and stability analyses are presented in the following sections.

Reach Determinations

6.28 Based on a review of the historic borings and the results of the Phase I CPT and Phase II SPT explorations, the channel was divided into reaches so conditions within each reach were relatively consistent and could be modeled using a single cross-section.

6.29 A total of six reaches were determined. The locations of the reaches and the analyzed cross-sections within each reach are shown on **Figures 6.2 and 6.3**. The floodwalls in Reaches 1 and 4 were not included in the stability analyses of the channel slopes but are discussed later in this report.

6.30 The individual reaches and the CPT and SPT borings considered for the reaches are shown in **Table 6.4** and discussed in more detail in the following paragraphs.

Table 6.4 Reach CPT/SPT and Station Limits

Reach No.	Station Limits	CPT/SPT
Reach 1	86+00 - 120+00	CPT-1, CPT-2, CPT-3, CPT-4, CPT-5, SPT-2, SPT-4, SPT-5
Reach 2	120+00 - 140+00	CPT-6, CPT-7, SPT-9
Reach 3	140+00 - 160+00	CPT-8, CPT-9, CPT-10, SPT-10, SPT-12, SPT-13
Reach 4	160+00 - 182+00	CPT-11, CPT-12, CPT-13, SPT-14
Reach 4.1	177+00	SPT-16
Reach 5	182+00 - 193+00	SPT-18

6.31 Reach 1 lies between Stations 86+00 and 120+00. Top of bank elevations in Reach 1 vary between approximately 33.0 and 40.0 feet. A sandy silt to silty clay layer tends to be present within the first 10.0 to 15.0 feet of Reach 1 soil profile. This initial layer is typically followed by a clay layer roughly 15.0 feet thick, which is then underlain by a slightly stronger clay layer to a depth of 40.0 feet.

6.32 Reach 2 lies between Stations 120+00 and 140+00, and the top of bank elevations range from elevation 40.0 to 53.0 feet. Typically, the soil profile in Reach 2 begins with a silty clay layer to approximately elevation 35.0 feet. A second layer of weaker clay is then encountered that ranged from 15.0 to 17.0 feet thick overlying a slightly stronger layer of clay and silty clay.

6.33 Reach 3 extends from station 140+00 to station 160+00, and the top of bank elevation ranges from elevation 53.0 to 61.0 feet. Reach 3 is distinguished due to a thick silty sand and sandy silt layer that typically extends to depths of 10 to 15 feet below the top of the bank. The initial layer is followed by a clay layer to elevation 21.0 feet. The final layer is a thin silty clay layer extending to elevation 13.0 feet.

6.34 Reach 4 extend from station 160+00 to station 182+00, and straddles the Montague Expressway. The top of bank elevation ranges from 61.0 to 65.0 feet. A stiff silty clay layer is usually encountered first, down to elevation 55.0 feet. This first layer is typically followed by a sandy clay layer that extends to elevation 33.0 feet, and is followed by a significantly stronger silty clay to sandy clay layer down to elevation 25.0 feet.

6.35 However, boring SPT-16 was within Reach 4 at the outside bend of the channel (Station 177+00) and this boring encountered much different conditions than the closest upstream and downstream borings. Boring SPT-16 encountered 4 feet of clayey sand fill at the ground surface overlying stiff clay to a depth of 12 feet. Below the stiff clay, 13 feet of soft to medium stiff clay was encountered to a depth of 25 feet. Because these soft to medium stiff clays could adversely impact the stability of the proposed slopes and because of its critical location at the outside bend of the channel, it was determined to analyze this section location. This analyzed section was designated as Reach 4.1.

6.36 Reach 5 extends from station 182+00 to station 193+00, and the top of bank elevation ranges from elevations 65.0 to 75.0 feet. An increasingly stiff clay and silty clay layer follows the first sand layer and extend to elevation 47.0 feet. The final layer is moderately stiff clay that typically extend down to elevation 30.0 feet.

Shear Strength Selections

6.37 Undrained Shear Strengths. To determine the undrained strengths of the cohesive soils on the project, SPT N-values, CPT relationships, and the results of the laboratory tests were all considered. However because the CPT testing provides a nearly continuous determination of the undrained strength of the soil with depth, the CPT data was evaluated first, then compared with the SPT and testing information.

6.38 For the CPT boring results, the undrained shear strength, s_u (Q-strength) is estimated with the following relationship:

$$s_u = \frac{q_t - \sigma}{N_{kt}}$$

where: s_u = undrained shear strength (psf)
q_t = total cone resistance (psf)

σ = overburden pressure (psf)
N_{kt} = dimensionless factor (10 to 18 but often 14 to 16)

6.39 Initially, the undrained shear strengths from the CPT borings were calculated using an N_{kt} value of 16. The results of the undrained shear strength determinations were then compared to the unconfined compression test results performed on two samples of the clays at the project. However, these two unconfined compression tests indicated undrained shear strengths of 623 and 721 psf, which were significantly less than the undrained strengths calculated for the CPT borings near these test locations. As a result, the undrained shear strengths from the CPT borings were recalculated using an N_{kt} value of 18.

6.40 For each reach, the undrained shear strengths from each CPT boring within that reach were plotted. The selected undrained strength was then conservatively selected based on an inspection of the plots for each reach. These plots of the undrained shear strengths from the CPT borings, unconfined compression tests, and our selected undrained strengths (Q-strengths) for the various clay layers in the five reaches are shown in **Figures 6.5 through 6.8**.

6.41 For the cohesionless sands on the project, the undrained strengths were assumed to be equal to the drained strengths. The drained strength determinations for the cohesionless sands are discussed in detail in the next section of the report.

6.42 The clayey sands on the project generally contained an appreciable amount of fines. It is believed that these cohesive sands will behave more similarly to cohesive soils rather than cohesionless soils. Therefore, to be conservative, the undrained strengths for the clays on the project were also assigned to the clayey sands.

6.43 For boring SPT-16, the undrained shear strengths for the clays were determined using the SPT N-values in accordance with the procedures outlined in Bowles (Bowles, 1997). The upper clay was assigned a cohesion value of 1,164 psf, the soft to medium stiff clays a cohesion value of 380 psf, and a cohesion value of 1,430 psf was determined for the underlying stiff clays. These calculations are presented in Appendix C. We would note that a shear strength test was assigned to a sample of the soft to medium stiff clay in this boring but the result of the test was very questionable and could not be used.

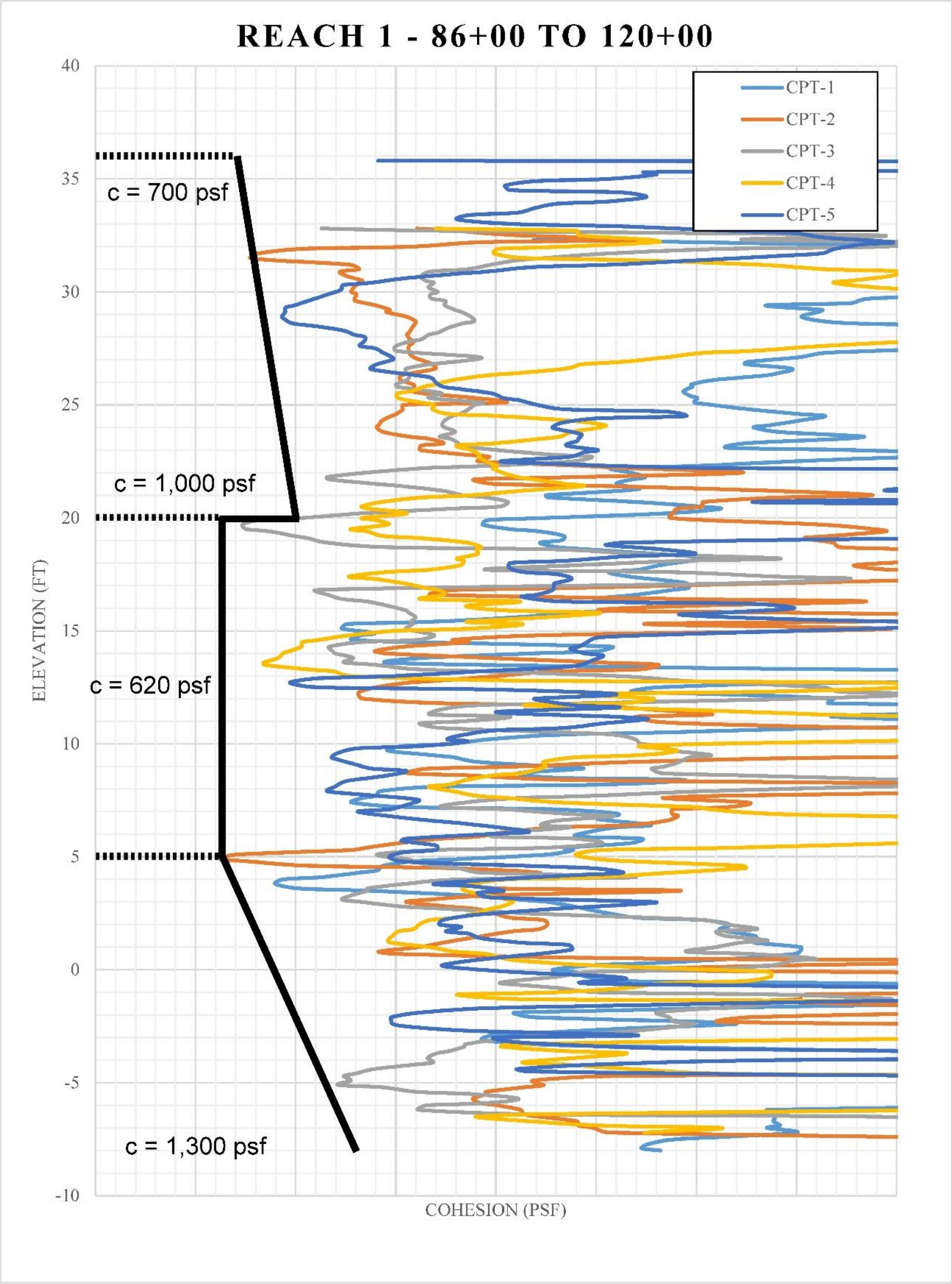

Figure 6.5 Reach 1 CPT Results and Selected Undrained Strengths

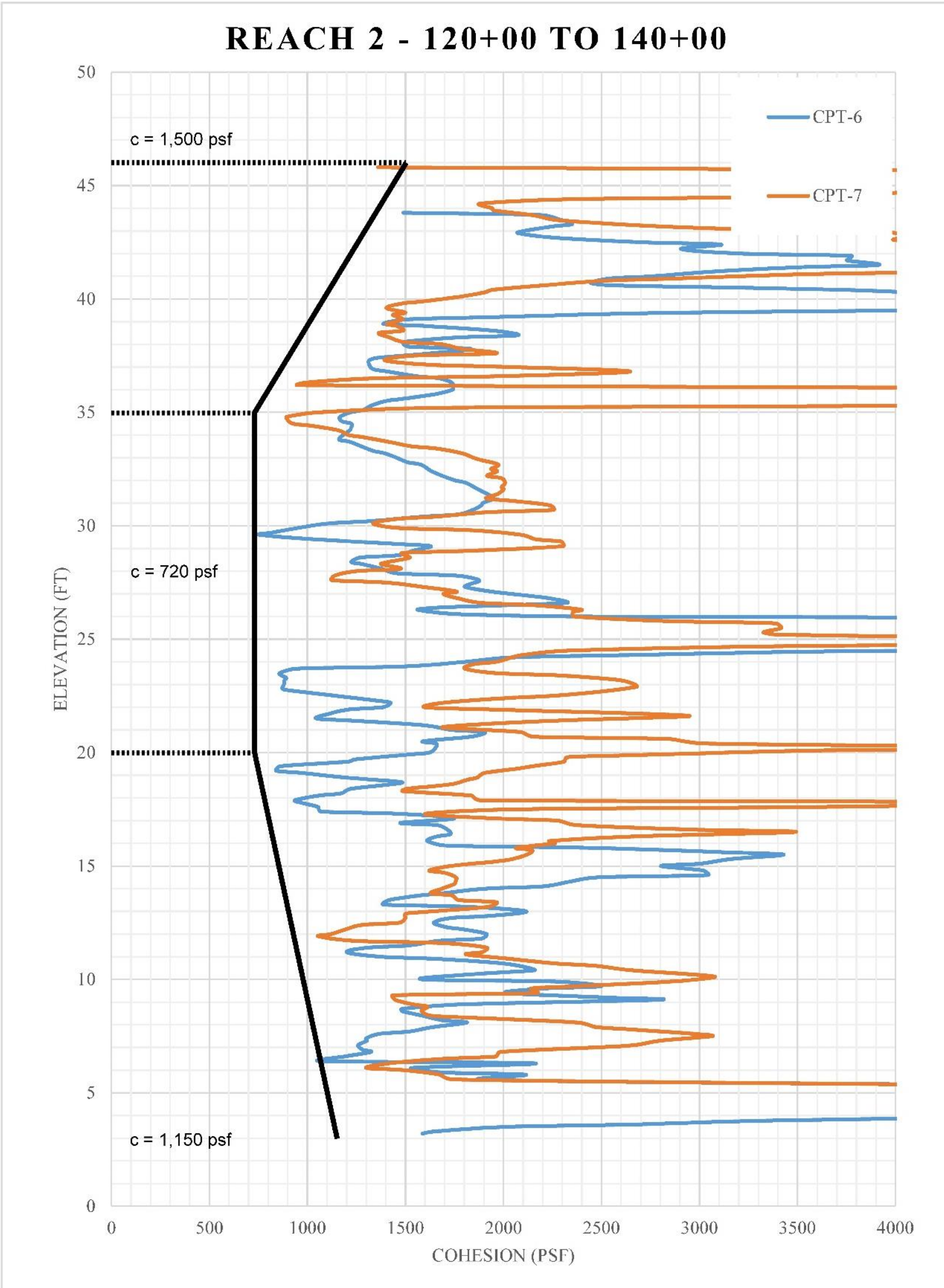

Figure 6.6 Reach 2 CPT Results and Selected Undrained Strengths

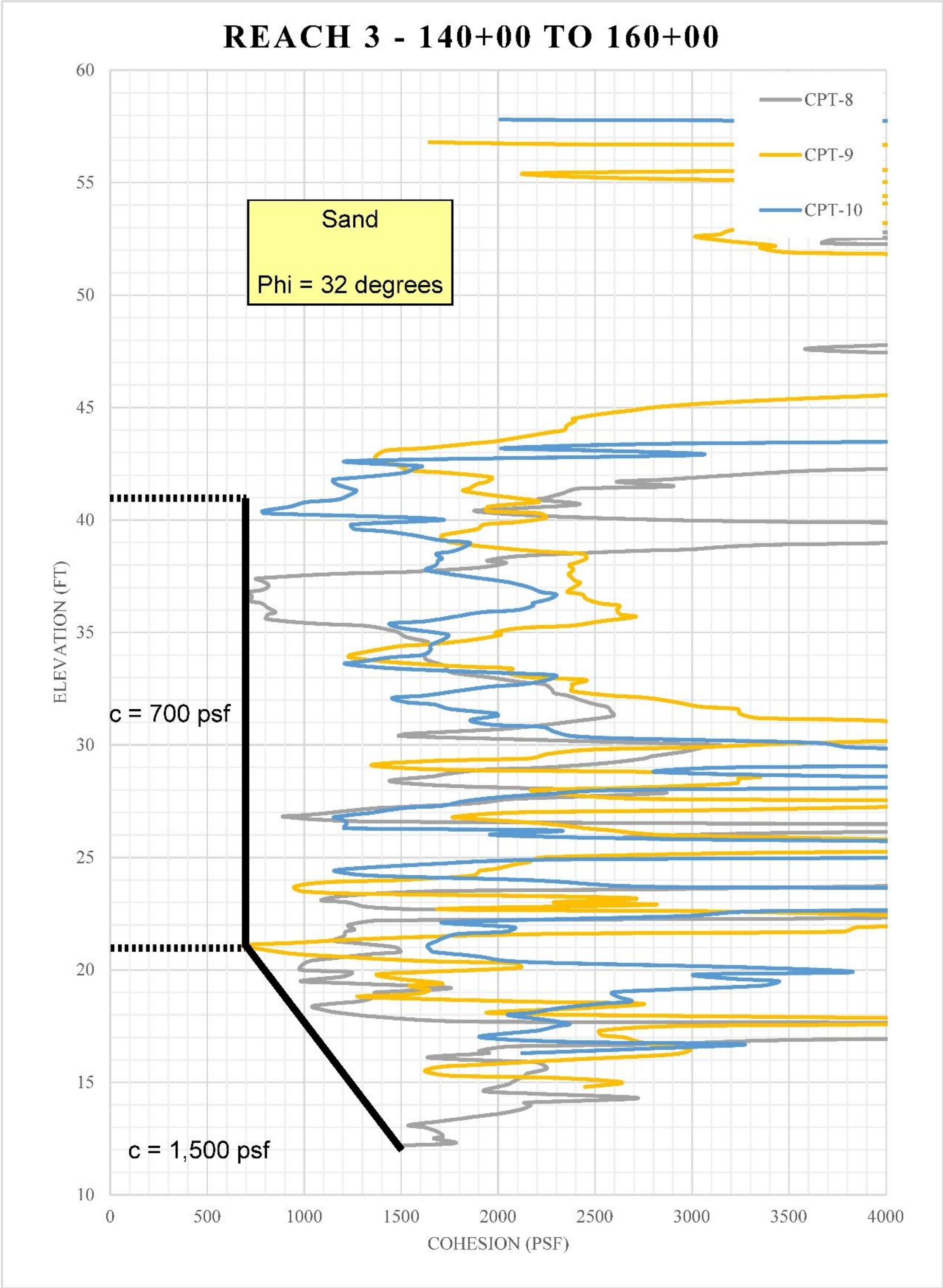

Figure 6.7 Reach 3 CPT Results and Selected Undrained Strengths

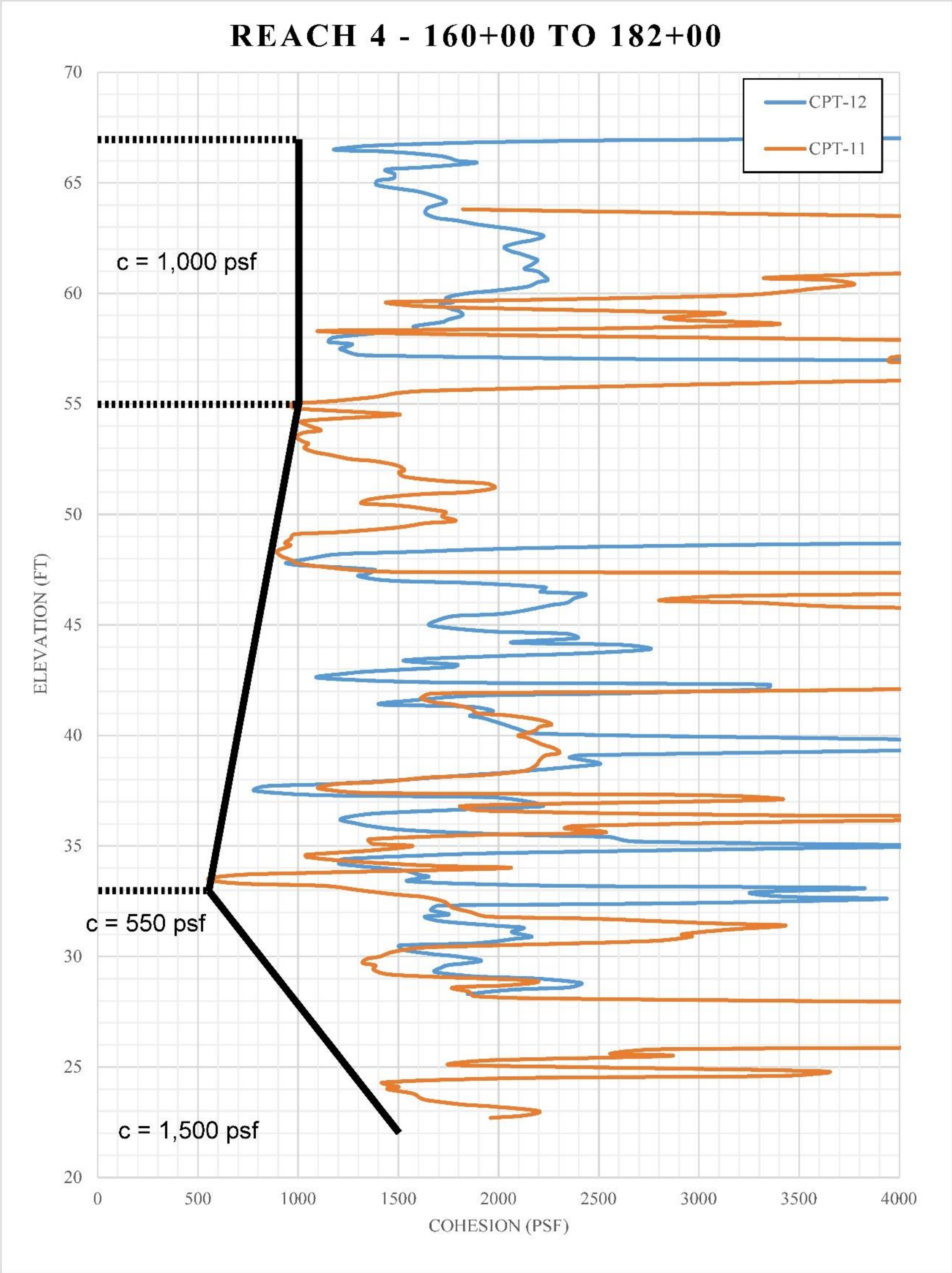

Figure 6.8 Reach 4 CPT Results and Selected Undrained Strengths

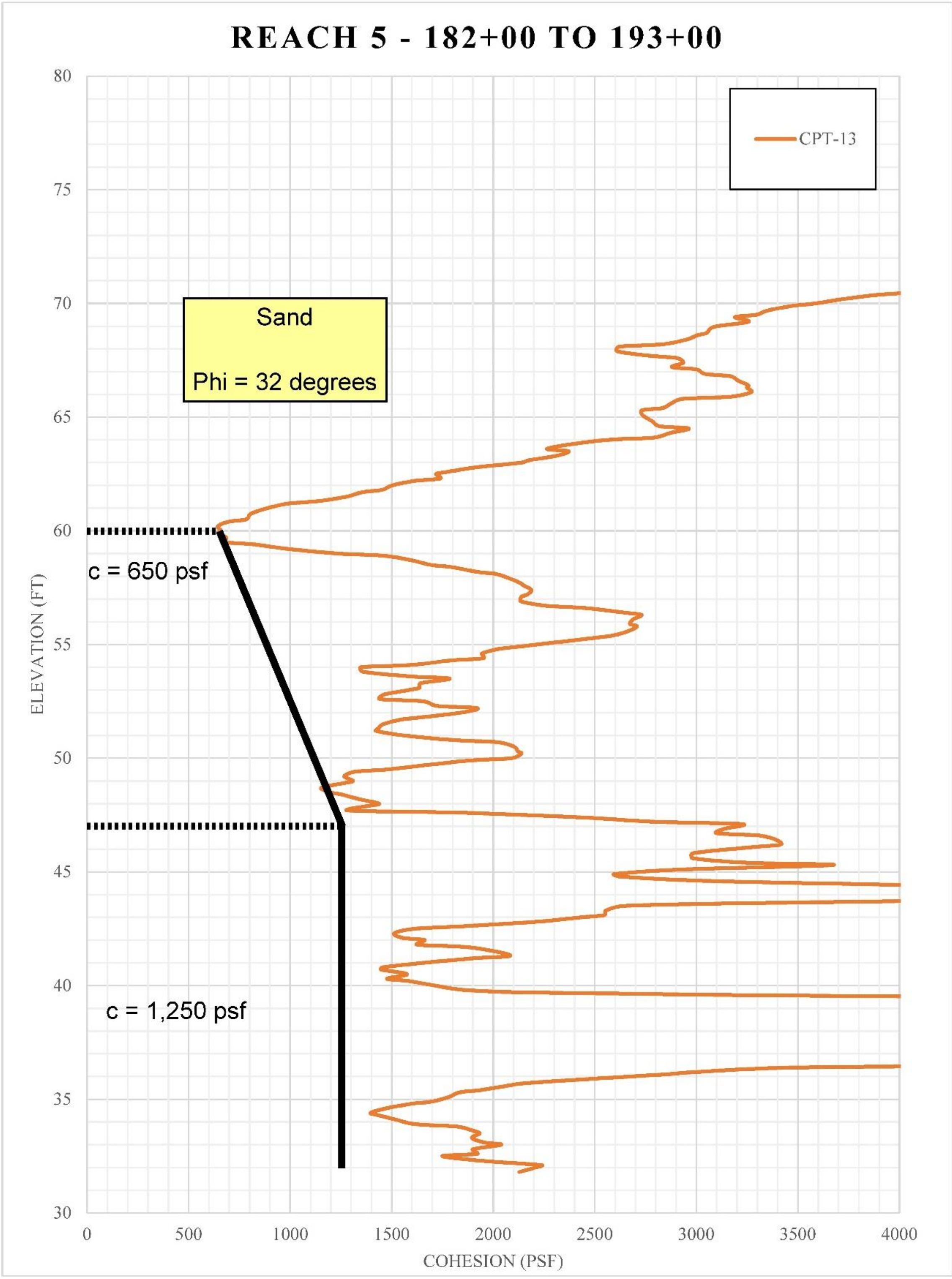

Figure 6.9 Reach 5 CPT Results and Selected Undrained Strengths

6.44 Drained Shear Strengths. The drained shear strengths (S-strengths) for the clays and sands in the channel slopes were selected based on the results of the classification of the soils, the SPT N-values, and two consolidated-undrained triaxial tests and the SPT results, respectively. For the clays, one of the triaxial tests was performed on a high-plastic clay with about 15 percent sand while the other was performed on a silty clay with about 45 percent sand. The drained strengths from the triaxial tests are listed below:

Silty clay (45% sand)	c' = 0 psf	ϕ' = 34.5°
High-plastic clay (15% sand)	c' = 180 psf	ϕ' = 30°

6.45 Based on these two results, the lower drained strengths (S-Strengths) of c' = 180 psf and ϕ' = 30° were selected for all of the clays on the project to be conservative. Based on our review of all of the borings and the laboratory test results, we believe these strengths are appropriate for all of the clays on the project, even the soft soils encountered in boring SPT-16.

6.46 The consolidated-undrained strengths (R-strengths) from these two tests varied significantly, likely due to the difference in sand content. The result for the silty clay (45 percent sand) was a c = 90 psf and a ϕ' = 18°. For the high-plastic clay (15 percent sand), the result was a c = 450 psf and a ϕ' = 12.5°. The lower of these two values is very low for clays and using the lower value was considered to be overly conservative. Therefore, these two strengths were averaged, and the average value was assigned to all of the clays resulting. Consequently, R-strengths of c = 270 psf and ϕ' = 15° were used for all of the clays on the project.

6.47 For the cohesionless, silty sands encountered on the project, the drained strengths were determined using the results of the SPT N-values obtained in the sands during the drilling operations. A review of the uncorrected N-values indicated a minimum value of 5 and an average value of 16.5. Using the relationship in Bowles (Bowles, 1997) that correlates uncorrected N-values to angles of internal friction in sands, friction angles of 32.5° and 35.8° were determined for the minimum N-value and average N-value, respectively. To be conservative, a friction angle of 32° was selected for all of the cohesionless sands on the project. These calculations are presented in Appendix C.

6.48 As mentioned in the discussion on the selection of undrained strengths, it is believed that the clayey sands will behave more similarly to cohesive soils rather than cohesionless soils. Therefore, to be conservative, the drained strengths for the clays on the project were also assigned to the clayey sands.

Stability Analyses

6.49 Method of Analyses. Slope stability analyses were performed using the slope stability analysis software Slide v.6.0. All analyses were performed using Spencer's method. Stability analyses were performed for the end-of-construction cases using Q-strength data and for the long-term cases using S-strength data. The drawdown cases were performed using the multi-stage, drawdown evaluations with composite S-strength and R-strength data in accordance with the procedures outlined in EM 1110-2-1902, Slope Stability (USACE, 2003).

6.50 Circular failure surfaces searches were performed for each analyzed cross-section and stability case. Based on our experience, non-circular failure surfaces are not as critical with the types of stratigraphies modeled at this project. However, this conclusion was confirmed by performing a non-circular failure surface search on the most critical cross-section and loading case determined by the results of the circular failure surface searches.

6.51 Cross-sections of the channel were based on the 60 percent design drawings. The proposed channel will be about 10 feet high with variable bottom widths feet. Side slopes of 2H: 1V were used but the rip rap and geocell slope protection were neglected to be conservative. Since the proposed slopes are 2H: 1V for the entire project, the critical cross-section locations were based on the height of the proposed banks. For Reaches 1 through 3, because they are relatively long reaches, two cross-sections were initially evaluated and the more critical selected for further analyses. In Reaches 4 and 5, because of their relatively short length, only a single, critical cross-section was selected. However, two cross-sections were analyzed in Reach 4 due to the conditions encountered in boring SPT-16, as described in previous sections. These cross-sections, with the results of the borings and the shear strength selections, were used to develop the analyzed sections. At each analyzed section, both banks were analyzed for stability. However, only the more critical of the banks is presented and discussed. The analyzed sections, along with the results of the stability analyses, are shown in Appendix C.

6.52 Load Cases Analyzed. As mentioned above, the stability of the channel slopes was performed in accordance with EM 1110-2-1902, Slope Stability (USACE, 2003). The load cases considered for the stability analyses are discussed below.

> **Case 1: End of Construction.** This case was evaluated for all of the analyzed sections. In this case, unconsolidated undrained (Q) strength parameters were used for this evaluation. The water level in the channel was assumed to be below the bottom of the proposed invert level. For this end-of-construction condition, this assumed water level is the most critical assumption since the water is the slope's stabilizing load.
>
> **Case 2: Steady State Seepage.** The stability analyses for the case of steady seepage were performed assuming the 100-year flood event is at that level for a long period sufficient to saturate the bank soils. This is a conservative assumption since it is anticipated that the 100-year event will not remain high enough for a sufficient period to saturate the bank soils. S-strengths were used for these analyses.
>
> **Case 3: Sudden Drawdown.** For the sudden drawdown analysis, it was assumed that the water level within the channel dropped from the 100-year level to near the bottom of the proposed channel. This is a very conservative assumption since it assumes the 100-year flood level will remain high enough in the channel to completely saturate the bank soils. In these analyses, the drained (S) strength parameters were used for the sand layers and the lower of the drained (S) and undrained (R) shear-strength envelopes was used for the clays. The staged drawdown feature of Slide v.6.0 was utilized and the program's documentation indicates that the procedure incorporated in the software matches the procedures outlined in EM 1110-2-1902, Slope Stability (USACE, 2003).

Case 4: Critical Flood Level. Finally, a critical flood analysis was performed on the reach cross-section that exhibited the lowest safety factor for Case 2, steady seepage at the 100-year flood level. For Case 4, steady seepage conditions and S-strengths were used. The critical flood level was found by varying the water level within the channel and determining which flood level resulted in the minimum safety factor. Since the other cross-sections exhibited higher safety factors for Case 2, if this case were run on the other cross-sections, they would exhibit safety factors greater than those determined for the critical cross-section.

6.53 Minimum Required Safety Factors. The required minimum safety factors used for each of the load cases was developed using the criteria in EM 1110-2-1902, Slope Stability (USACE, 2003). Table 3-1 in the EM presents the required minimum safety factors for new embankment dam slopes. However, in Section 3-4 of the EM, there is discussion of the minimum required safety factors to use in the stability analyses of other slopes. Within paragraph 3-4, the EM states:

…Typical minimum acceptable values of factor of safety are about 1.3 for end of construction and multistage loading, 1.5 for normal long-term loading conditions, and 1.1 to 1.3 for rapid drawdown in cases where rapid drawdown represents an infrequent loading condition. In cases where rapid drawdown represents a frequent loading condition, as in pumped storage projects, the factor of safety should be higher.

6.54 Based on this guidance, required minimum safety factors of 1.3, 1.5, and 1.3 were selected for the end of construction case, the long-term 100-year flood level steady seepage and critical flood steady seepage cases, and the rapid drawdown case, respectively. We believe the rapid drawdown case may be a relatively frequent loading condition in the channel, so a higher required minimum safety factor should be considered for this case.

6.55 Analyses Results. The results of the stability analyses are summarized in **Table 6.5** and presented in Appendix C. As can be seen in the table, the calculated critical safety factors were all above the required minimum safety factors.

Table 6.5 Summary of Stability Analyses Results

Reach	Case Analyzed	Critical F.S. (Req'd min.)
Reach 1 (86+00 to 120+00)	End of Construction (Q)	2.44 (1.3)
	Steady Seepage (S)	3.05 (1.5)
	Sudden Drawdown (R,S)	1.61 (1.3)
Reach 2 (120+00 to 140+00)	End of Construction (Q)	2.68 (1.3)
	Steady Seepage (S)	2.61 (1.5)
	Sudden Drawdown (R,S)	1.62 (1.3)
Reach 3 (140+00 to 160+00)	End of Construction (Q)	2.26 (1.3)
	Steady Seepage (S)	2.19 (1.5)
	Sudden Drawdown (R,S)	1.40 (1.3)
Reach 4	End of Construction (Q)	2.69 (1.3)

Reach	Case Analyzed	Critical F.S. (Req'd min.)
(160+00 to 182+00)	Steady Seepage (S)	2.69 (1.5)
	Sudden Drawdown (R,S)	1.73 (1.3)
Reach 4.1 (SPT-16 at 177+00)	End of Construction (Q)	2.41 (1.3)
	Steady Seepage (S)	3.07 (1.5)
	Sudden Drawdown (R,S)	1.93 (1.3)
Reach 5 (182+00 to 193+00)	End of Construction (Q)	1.44 (1.3)
	Steady Seepage (S)	1.69 (1.5)
	Sudden Drawdown (R,S)	1.42 (1.3)
Critical Drained Section	Critical Flood	1.65 (1.5)
Critical Undrained Section	End of Construction (Q)	4.50 (non-circular) (1.3)

6.56 For Reaches 3 and 5 where sand was present in the proposed channel slope, the critical safety factors were infinite-slope type failures with safety factors of 1.2 or greater. Infinite-slope type failures represent a theoretical minimum safety factor but the failure surfaces are very shallow, raveling-type of surfaces that are maintenance issues and do not impact the integrity of the slope. Typically, a safety factor greater than 1.0 for an infinite-slope type failure is considered acceptable. For cases where an infinite-slope type surface was the critical failure surface and the safety factor was greater than 1.0, deeper surfaces were analyzed to determine a more appropriate safety factor. This analysis was used to confirm that more realistic failure surfaces had safety factors greater than the required minimum.

1.5H: 1V Slopes

6.57 It is understood that steeper slopes of up to 1.5H: 1V may be required in isolated areas to maintain the channel capacities, such as at bridges or other channel constrictions. If 1.5H: 1V slopes must be used in an area, we recommend that these slopes be constructed with rip rap or channel protection stone. If an encroachment into the channel is prohibitive, this may require overexcavating the soil into the bank, then rebuilding the slope with the rip rap or channel protection stone. The toe of this rock zone should be keyed into the channel bottom to provide stability. Stability analyses would be needed to determine the proper configuration and amount of rip rap or channel protection stone to use, but it is anticipated that a slope 10 feet high would require a rock zone that was a few to several feet thick for adequate stability.

UPRR TRESTLE and other CULVERT DESIGNs

General

6.58 As mentioned earlier in this report, current plans call for the demolition of the existing UPRR timber trestle bridge over Berryessa Creek and replacement with a two-cell reinforced concrete culvert. The UPRR culvert project extends from channel station 160+44 to 161+46. In addition, new culverts are planned for lateral drainage features entering the channel at Los Coches Avenue and Piedmont Avenue.

6.59 Preliminary plans indicate that the proposed UPRR culvert will be a double, 10-foot wide (W) and 9-foot high (H) reinforced concrete box (RCB) structure. The culvert invert elevation is anticipated to range from elevation 49.25 to 49.67 feet, which is approximately one foot below the lowest current invert elevation in the existing creek.

6.60 The proposed culvert at Los Coches is a 14-foot wide (W) and 6-foot high (H) reinforced concrete box (RCB) structure. The culvert invert elevation is anticipated to range from elevation 19.92 to 33.23 feet.

6.61 The proposed culvert at Piedmont is a 14-foot wide (W) and 6-foot high (H) reinforced concrete box (RCB) structure. The culvert invert elevation is anticipated to range from elevation 26.21 to 30.71 feet.

Foundation Preparation

6.62 Based on subsurface conditions encountered in the exploratory borings and on potential high groundwater conditions it is anticipated that saturated, clayey soils could be encountered at the proposed base of culvert elevations. It is expected that these conditions will produce a relatively soft bearing surface and difficult working conditions. Therefore, it is recommended that an engineered fill mat be constructed within the area below the proposed culverts and any appurtenant wing wall footings. The engineered fill should be constructed as follows:

- Over-excavate at least 2 feet below the base of the culvert slab or wall footing elevation.
- At the UPRR culvert location, cut and remove all existing pile foundations for the exiting trestle at a depth of at least 6 inches below the excavated surface.
- If necessary, stabilize the soft subgrade by working open-graded aggregate material (typically ¾-inch or 1.5-inch crushed rock, coarser for softer subgrade) at least 4 to 6 inches into the soil.
- Place non-woven geotextile, Mirafi 180N or approved equivalent, over the stabilized subgrade.
- Place and compact well-graded select fill. The fill can be either Crushed Aggregate Base or Crushed Miscellaneous Base to specified compaction over the geotextile.

Culvert and Retaining Wall Backfill

6.63 It is expected that due to the clayey nature of most of the on-site material, it will not be suitable as a backfill immediately behind site retaining walls. Free draining material should be used for backfill behind retaining walls. Consequently, an approved import material should be used for the backfill within at least 2 feet behind the back side of the wall. Suitable material should have a Sand Equivalent of about 30, an Expansion Index of less than 20, and fines content (passing #200 sieve) of less than 15 percent. The suitability of the import material for retaining wall backfill should be verified at the time of construction.

6.64 The backfill should be moisture-conditioned to at least optimum moisture content and compacted in loose horizontal lifts not more than 8 inches in uncompacted thickness to at least 90 percent of the maximum dry density as evaluated by the latest version of ASTM D 1557.

Subdrainage

6.65 Retaining walls should be constructed to limit potential for hydrostatic pressure built up behind the wall by installing subdrains near the base of the wall. The drain pipe should consist of a minimum 4 inch diameter perforated PVC pipe surrounded by 2 cubic foot per foot of the Class II Permeable Material (Caltrans Standard Specifications - Section 68), or by ¾ inch crushed rock wrapped in suitable non-woven filter fabric, e.g., Mirafi 140NL or approved equivalent. Perforations in the drain pipe should have a maximum diameter of 1/4 inches or 3/8 inches for Class 2 Permeable or 3/4 -inch crushed rock drain material, respectively, spaced 3 inches on center, and be arranged in 2 rows at a radial spacing of approximately 120 degrees. The axis of the included angle between the perforation rows should be positioned downward to form a flowline. The drain pipe should discharge through a solid pipe to appropriate outlets, such as the storm drain system or through the wall. The maximum length of the drain pipe between discharge outlets should not exceed 200 feet.

6.66 Unless the culvert designs include lateral and uplift pressures for hydrostatic forces, continuous subdrains should also be installed behind the base of the culvert walls. If the UPRR, Los Coches, and Piedmont culverts are being designed to resist uplift pressures, a groundwater elevation of +55, +30, and +35 feet, respectively, should be utilized.

Settlement

6.67 Based on the consolidation testing of the saturated clayey foundation soil underlying the UPRR culvert (Station 161+00), it is expected that some long-term settlement of the culvert will occur. The total settlement at the midpoint of the culvert is estimated to be approximately 1.5 inches. This amount of settlement is not expected to be problematic to the structure or rail subgrade, however, it is recommended that a camber in the UPRR culvert invert incorporate this amount of potential differential settlement from the ends to the midpoint of the culvert. Grading provisions above the UPRR culvert should incorporate this amount of potential settlement at the centerline of the channel.

6.68 Settlements of the other two culverts (Piedmont Creek and Los Coches Creek), wing walls, or retaining structures placed on foundation soils prepared in accordance with Section 9.2 "Foundation Preparation" are estimated to be less than 1 inch.

Design Parameters

6.69 The culverts and appurtenant retaining walls may be designed using the following parameters. These design values are based on foundation preparation and grading recommendations presented in this report.

Vertical Loading

6.70 Vertical loads on the UPRR culvert should be assessed by the design chart presented in Figure 5.2 of USACE EM 1110-2-2902 "Engineering and Design, Conduits, Culverts and Pipes" for railroad loading and Figure 8-16-1 in the AREMA Manual for Railway Engineering Chapter

8. Both charts should be consulted for this culvert because, depending on embedment depth, total loading varies between the two charts. Based on maximum density testing of on-site soils, the dead load curve for both design charts should be adjusted to reflect a total unit weight of 130 pcf. Vertical loads on the Los Coches and Piedmont culverts should be assessed by the design chart presented in Figure 5.2 of USACE EM 1110-2-2902 "Engineering and Design, Conduits, Culverts and Pipes."

6.71 If the UPRR, Los Coches, and Piedmont culverts are being designed to resist uplift pressures, a groundwater elevation of +55, +30, and +35 feet, respectively, should be utilized.

Lateral Loading

Retaining Walls

6.72 Retaining walls should be designed for the appropriate lateral earth pressure based on the following design parameters and equivalent fluid pressures (**Tables 6.6 and 6.7**):

Table 6.6 Retaining Wall Design Parameters

Active Earth Pressure Coefficient	0.39
At-Rest Earth Pressure Coefficient	0.56
Allowable Passive Pressure Coefficient	1.7
Allowable Friction Coefficient	0.30
Total Unit Weight	130 pcf
Buoyant Unit Weight (below groundwater)	67.6 pcf

Note: Assumes level backfill behind the wall

Table 6.7 Equivalent Fluid Pressures[1]

Description	Above Water Table (pcf)	Below Water Table (pcf)[2]
Active Equivalent Earth Pressure	51	26
At-Rest Equivalent Earth Pressure	73	38
Passive Equivalent Earth Pressure	221	115

Note: (1) Assumes level backfill behind the wall
(2) Soil pressure only

6.73 Determining whether the active or at-rest condition is appropriate for design will depend on the flexibility of the walls. In clayey soils walls that are free to rotate at least 0.01 radians (deflection at the top of the wall of at least 0.01 x H) may be designed for the active condition. Walls that are not capable of this movement should be assumed rigid and designed for the at-rest condition. The effect of any surcharge (dead or live load) located within a 1(H):1(V) plane drawn upward from the heel of the wall footing should be added to the lateral earth pressures by multiplying the surcharge pressure by the appropriate earth pressure coefficient.

6.74 Where design requires that seismic earth forces be considered, the following appropriate seismic earth forces should be utilized (**Table 6.8**).

Table 6.8 Summary of Seismic Earth Forces

Seismic Earth Force (100 year return period)	$17.6H^2$ lbs/foot of wall
Seismic Earth Force (144 year return period)	$20.0H^2$ lbs/foot of wall
Seismic Earth Force (475 year return period)	$30.7H^2$ lbs/foot of wall
Seismic Earth Force (949 year return period)	$37.1H^2$ lbs/foot of wall
Seismic Earth Force (MCE or 2475 year return period)	$24.4H^2$ lbs/foot of wall

Seismic earth force should be applied at a distance of 2/3H up from the base of the wall.
H = Height of Wall (feet)

Culverts

6.75 For culvert design, the AREMA manual requires that minimum and maximum earth pressure coefficients of 0.33 and 1.0, respectively, be used to evaluate lateral pressure on the structure. We recommend that the Los Coches and Piedmont culverts be designed using the same earth pressure coefficients. Vertical pressures used in the calculations should be those calculated by the design charts discussed in Section 9.7, Vertical Loading. If the UPRR, Los Coches, and Piedmont culverts are being designed to resist uplift pressures, a groundwater elevation of +55, +30, and +35 feet, respectively, should be utilized.

Bearing Capacity

6.76 Design of culverts and footing foundations invert slabs of the for retaining walls should be based on an allowable bearing capacity defined by the following equation:

q_{all} = 1120 + 260D + 60B (psf) (3,000 maximum)
q_{all} = allowable bearing pressure
D = minimum footing embedment (feet)
B = footing width (feet)

6.77 The allowable bearing pressure may be increased by one-third when considering live loads and seismic loads.

6.78 The modulus of subgrade reaction for the design of the culvert slabs can be calculated as:

$$K_s = \frac{280}{B} \ in \ pci$$

where B is the governing width of the element in feet, but no more than 14 times the thickness of the element.

Cutoffs

6.79 The upstream and downstream edges of the culvert slab/apron should include a full width cutoff wall extending at least 3 feet below the base of the slab or at least 6 inches below the potential scour depth, whichever is deeper.

FLOODWALLS

General

6.80 Based on the design drawings, it appears that a short floodwall is needed on the left bank to contain the channel flows and an adequate freeboard between Stations 103+50 and 115+23 and Stations 171+00 and 175+50. The floodwall will only be a few feet high at the most per the 60 percent drawings.

6.81 The two SPT borings in the area of the floodwall between Stations 105+00 and 115+43 (SPT-4 and SPT-5) encountered 3 feet of uncontrolled clay fill at the ground surface. This uncontrolled fill is not considered suitable to support the proposed floodwall. Therefore, it is recommended that this fill be over excavated, replaced, and recompacted beneath the floodwall or the floodwall should be founded in the natural clays below the fill. If over excavation and replacement is performed, it is possible that the existing material can be reused as fill, based on the classifications of the material encountered in the borings; however, this will have to be confirmed in the field during construction. Any fill placed to support the floodwall should be placed in 8-inch thick loose lifts and compacted to at least 95 percent of the material's maximum dry density as determined by ASTM D 1557.

6.82 The floodwall between Stations 171+00 and 175+50 lies between an existing building and the top of the channel bank. To construct the floodwall, the existing material behind the building will be overexcavated about 5 feet to construct the floodwall. Following the floodwalls construction, the overexcavated material will be replaced to the original grade. Because the floodwall is essentially buried within the soil, the net load on the foundation soils beneath the floodwall will be very low.

6.83 The floodwalls should be designed in accordance with the following Corps' Engineering Regulations and Engineering Manuals:

ER 1110-2-1150 Engineering and Design for Civil Works Projects
ER 1110-2-1806 Earthquake Design and Evaluation for Civil Works Projects
EM 1110-2-2100 Stability Analysis of Concrete Structures
EM 1110-2-2104 Strength Design for Reinforced-Concrete Hydraulic Structures
EM 1110-2-2502 Retaining and Flood Walls

Earth Pressures and Uplift

6.84 Most of the load on the floodwalls will be from hydrostatic loads from channel flows. If earth pressures are needed for the structural design, the values listed in **Tables 6.6 and 6.8** should be used.

6.85 Cohesive soils should be assumed for the backfill around the floodwalls. Granular material should not be used for backfill unless needed for seepage control at the landside toe of the floodwall. However, any seepage relief needs to be analyzed and designed for appropriate exit gradients.

6.86 The floodwall design should also account for uplift on the base of the foundation. The uplift should vary linearly from the heel to the toe of the wall. The uplift pressure value at the heel should be equal to the full hydrostatic pressure from the flood level while the uplift pressure value at the toe should be equal to the full hydrostatic pressure from the tailwater level.

Sliding

6.87 Based on the results of the borings, the proposed floodwalls should bear on clay soils. For concrete on clay soils, it is recommended that a friction factor of 0.30 be used to determine the sliding factor of safety along the base of the walls.

Bearing Capacity

6.88 The allowable bearing capacities of the floodwall foundations were determined using the procedures in EM 1110-1-1905, Bearing Capacity of Soils. The undrained strengths from the borings along the floodwall were used, and Meyerhof's equation was considered. The calculations indicate an allowable undrained bearing capacity of the soils beneath the floodwall equal to 1,250 psf. It was assumed that the floodwall alignment in relation to the slope was as shown in the 60 percent design drawings. The undrained bearing capacity calculations for the floodwall are presented in Appendix C.

6.89 The allowable bearing capacity of the soils should be calculated based on both undrained and drained strengths. However, the bearing capacity calculation using drained strengths requires the dimensions of the floodwall foundation, which will not be finalized until the 90 percent design. However, we estimated a minimum floodwall foundation width assuming a head differential of 2 feet and an embedment of 2 feet. Using the line of creep analysis presented in EM 1110-2-2502, the calculations indicate that a minimum floodwall foundation width of 4.5 feet should be considered.

6.90 Once the 90 percent floodwall design is complete and the design and the foundation dimensions are known, the allowable bearing capacity of the soils using drained strengths should be checked. In addition, the line of creep analysis should be reviewed to determine that the foundation width and embedment are sufficient to provide an adequate safety factor against piping.

Settlement

6.91 If the floodwalls are designed for the allowable bearing capacity recommended in the previous section, we estimate that the floodwall total settlements will be less than one inch. Differential settlement between floodwall monoliths should be less than 0.5 inches. However, once the floodwall is completed to the 90 percent level, this should be confirmed by checking the settlement based on the final dimensions and actual bearing pressures of the foundation.

TRANSITION STRUCTURES

6.92 Transition structures will be constructed at several locations along the channel. In the design drawings, transition structures are located at each of the bridge crossings except for Yosemite Drive and Ames Avenue. Based on our review of the design and the boring results, we see no significant geotechnical impacts on the design or construction of the transition structures with the exception of the transition structure beneath the Los Coches Avenue Bridge.

6.93 The Los Coches Avenue Bridge was constructed in the mid-1960s and is currently the responsibility of the City of Milpitas. The structure is a two-span bridge with the abutments and pier supported on driven, pre-cast concrete piles. Based on as-built drawings of the bridge, the piles were roughly 50 feet long and designed for an axial capacity of 45 tons.

6.94 The excavation for the transition structure beneath Los Coches Avenue will remove soil from in front of the abutment piles and could reduce the axial and lateral capacity of the abutment piles. Cursory estimates of the structural loads on the piles indicate that the abutment foundations do not meet current standards in terms of axial resistance or lateral resistance.

6.95 Based upon a preliminary analysis of the bridge, and the understanding that the bridge does not meet the current loading requirements, a slight reduction in axial capacity will not significantly affect the conclusions and/or performance of the bridge.

6.96 In addition, with the soil in front of the abutment piles removed, resistance to lateral loading could be slightly reduced. The magnitude of this deflection cannot be accurately determined without a very detailed structural study of the bridge. However, the abutment deflection could impact the transition structure and possibly damage or crack the transition structure. Therefore, it is recommended that the transition structure beneath Los Coches be designed to accommodate some movement from the bridge abutment piles.

6.97 With the assumption that the bridge will act like a concrete diaphragm and will transfer the longitudinal seismic load into the soil behind the abutment. The at-rest resistance will be engaged at the onset of any movement. The onset of movement in this case will be defined as equivalent to 0.1% of the abutment height. Because of the small amount of movement required to engage the at-rest resistance, we will assume to full depth of the abutment will push against the soil backfill. With an abutment height of 8.6 feet, and the calculated movement based on 0.1% of the height, a value of 0.1 inches has been estimated as the movement needed to engage the at-rest resistance. The at-rest resisting value has been calculated as 4,440 pounds per foot of abutment width. The as-built plans show the bridge abutment width at 48 feet, which equates to a total resistance of 213 kips. This exceeds the estimated imposed seismic load of 199 kips.

6.98 Since the abutment needs to move approximately 0.1 inches to engage the at-rest pressure, the piles would also need to move this distance. At this amount of deflection, the existing piles would see an approximate maximum load of 6.6 kips of shear and a moment of 9.92 kip-ft. These values were provided and calculated by an L-Pile program. Based the analysis, the existing piles should each be adequate for approximately 8 kips of shear and 22 kip-ft of moment, which both exceed the calculated values for a deflection of 0.1 inches. Assuming the existing piles obtain their vertical capacity through skin friction, the pile vertical capacity would probably be reduced by about 6% due to the removal of some soil in front of the piles. We do not believe that this amount of reduction in the vertical pile capacity is significant enough to compromise the vertical load carrying capacity of the piles. A letter regarding the additional analysis was prepared and submitted to the City of Milpitas for their review as part of the permitting process through the City. This letter and analysis is included in the structural appendix of this DDR.

6.99 The modulus of subgrade reaction for the design of the transition slabs can be calculated as:

$$K_s = \frac{240}{B} \; in\, pci$$

where B is the governing width of the element in feet, but no more than 14 times the thickness of the element. This K_s value is less than that used for the culvert slabs since the transition slabs do not exert a significant load on the subgrade and soft soils beneath the transition slabs may not be removed during construction.

6.100 Due to the potential for the presence of granular layers near the channel invert, it is recommended that the cut off walls at the upstream and downstream ends of the transition structures be extended to a depth of 4 feet below the channel invert. Due to the corrosive nature of the project's soils, it is recommended that concrete cut off walls be used rather than sheet pile walls.

SCOUR AND EROSION PROTECTION

6.101 It is understood that rip rap will be used for scour protection near the base of the slopes along the channel. Rip rap is also being used for the channel invert between approximately Stations 115+00 and 164+00. The rip rap material size and toe-down depth should be designed in accordance with EM 1110-2-1601 and ETL 1110-2-120.

6.102 It is anticipated that the rip rap will be imported to the site from commercial sources. The construction documents should require the contractor to provide rip rap from only qualified and approved sources that meet the requirements of the Corps and the California Department of Transportation. The commercial source used to prepare the construction cost estimate was the Lake Herman Quarry in Vallejo, California. The phone number for this quarry is 707-643-3261.

SOIL CORROSIVITY

6.103 Laboratory testing was performed on representative soil samples to evaluate soil corrosivity to buried steel and concrete. **Table 6.9** presents the results of the corrosivity testing.

Table 6.9 Corrosivity Test Results

Location	**Sample ID**	**Depth (feet)**	**pH**	**Minimum Resistivity (ohm-cm)** CTM 643	**Chloride Content** CTM 422	**Soluble Sulfate Content** CTM 417
SPT-4	SK-1	0 – 5	7.7	1,160	0.0025%	0.0092 %
SPT-5	SK-1	0 – 5	7.8	1,274	0.0023%	0.0270 %
SPT-12	SK-1	0 – 5	7.3	488	0.0084%	0.0566 %
SPT-12	SPT-8	17.5 – 19	7.7	1,908	0.0022%	0.0032 %
SPT-13	SK-1	0 – 5	7.7	910	0.0036%	0.0124 %
SPT-13	SPT-6	12.5 – 14	8.0	3,116	0.0006%	0.0019 %
SPT-16	SPT-1	2 – 3.5	7.6	2,388	0.0004%	0.0057 %
SPT-18	SK-1	0 – 5	7.9	2,228	0.0004%	0.0057 %

6.104 Per CBC 2013/ IBC 2012, Section 1904.3, concrete subject to exposure to sulphates shall comply with the requirements set forth in ACI 318, Section 4.3. Based on the measured water-soluble sulphate results, the exposure of buried concrete to sulphate attack should be considered “not applicable," i.e., exposure class S0 per ACI 318, Table 4.2.1. Consequently, injurious sulfate attack is not a concern for concrete with a minimum 28-day compressive strength of 2,500 psi.

6.105 Per CBC 2013, Section 1904.4, concrete reinforcement should be protected from corrosion and exposure to chlorides in accordance with ACI 318, Section 4.4.

6.106 The minimum soil resistivity values indicate that on-site soils have a high to very high metallic corrosion potential.

PAVEMENT DESIGN PARAMETERS

General

6.107 Access roads are planned along both sides of the proposed channel for inspection and maintenance purposes. However, the type of roadway surface has not been determined at this time. General recommendations for the proposed access roads construction and design are presented below.

Subgrade Design

6.108 Based on the results of the laboratory testing, it is recommended that the proposed access road pavements be designed based on an R-value of 8. This recommendation assumes that pavement subgrades are prepared and constructed as recommended in the following section.

Subgrade Construction Recommendations

6.109 The proposed access roads subgrade should be stripped of all topsoil or organic soils to a point 5 feet outside of the roadway limits but not outside the project right-of-way. Once the subgrade is cut to grade, it should be proofrolled with heavy construction equipment and any areas that pump or deflect excessively should be overexcavated. After proofrolling, the subgrade should be compacted then scarified to ensure a good bond with the initial fill lift.

6.110 The fill beneath roadways should be spread in 8-inch thick loose lifts and uniformly compacted with a sheeps-foot-type roller to 95 percent of the material's maximum dry density (ASTM D 1557). The moisture content of the fill should be within 3+ percent of the material's optimum moisture content.

OTHER CONSTRUCTION RECOMMENDATIONS

Site Preparation and Fill Placement

6.111 The surface should be cleared of any topsoil, pavement, structures, vegetation, trash, and debris prior to commencement of any earthwork or foundation construction. Any subterranean installations such as pipes, utility collectors, tanks, etc. that are not to be preserved should be abandoned per the geotechnical engineer’s recommendations and in accordance with applicable regulations.

6.112 Based on the design cross-sections, some areas will require small slivers of fill to be placed on existing slopes. Where new engineered fill will be placed on an existing slope, the fill should be supported by a shear key constructed at the base of the toe of slope. The key should extend to a minimum depth of 3 feet below existing grade, have a minimum bottom-width of 5 feet, and side slopes of 1H: 1V.

6.113 In addition, existing slopes to receive fill must be benched with 2-foot-high vertical cuts prior to fill placement. In order to adequately compact the face of fill slopes, it is recommended that the fill slopes be overbuilt by a foot and trimmed back to the final configuration.

6.114 Fill should be placed in horizontal lifts not more than 8 inches in loose, uncompacted thickness. All fill placement associated with the replacement of the excavated soils, or fill placed to achieve finished grade or subgrade should be moisture-conditioned to within 3+ percent of the optimum moisture content and compacted to at least 92 percent of the maximum dry density, as evaluated by the latest version of ASTM D 1557. However, fill placed below pavements should be compacted to at least 95 percent of the maximum dry density, as evaluated by the latest version of ASTM D 1557.

6.115 Based on the findings from the borings, it appears that most of the excavated on-site soils may be re-used as compacted fill provided they are free of organics, deleterious materials, debris, and particles more than 3 inches in largest dimension. Locally, particles up to 4 inches in largest dimension may be incorporated in the fill soils.

6.116 However, it should be noted that softer, wetter soils were encountered near the existing channel invert. These soils may need to be spread, disked, and dried before they can be used for fill.

6.117 Specifically, an area of note was in the vicinity of boring SPT-16 (Station 177+00), which encountered about 13 feet of soft to medium stiff clay near the existing channel invert. It may be difficult to excavate these soft soils and special efforts or equipment may be required to remove these soils. It is anticipated that these soils will not be suitable for reuse as fill without drying significantly.

Temporary Excavation and Construction Slopes

6.118 The on-site soils are not expected to pose unusual excavation difficulties, and therefore, conventional earth-moving equipment may be used. Localized sloughing/raveling of exposed soil intervals should be anticipated. All excavations should be performed in accordance with California Division of Occupational Safety and Health (CalOSHA) regulations. The on-site soils above the groundwater level may be considered a Type B soil, as defined by the current CalOSHA soil classification system.

6.119 Unsurcharged excavations: Temporary short-term, generally less than five days, unsurcharged excavations shallower than 4 feet may be excavated with vertical sides. Sides of temporary, unsurcharged, excavation deeper than 4 feet should be sloped back at an inclination of 1H: 1V or flatter. Where space for sloped sides is not available, shoring will be necessary.

6.120 Surcharge setback recommendations: Stockpiled (excavated) materials should be placed no closer to the edge of a trench excavation than a distance defined by a line drawn upward from the bottom of the trench at an inclination of 1(H):1(V) but no closer than 4 feet. A greater setback may be necessary when considering heavy vehicles, such as concrete trucks and cranes. Alternatively, a shoring system should be designed to allow reduction in the setback distance.

6.121 Excavation below groundwater: The on-site soils below the groundwater level should be considered a Type C soil. It should be anticipated that excavation at or below the current creek level will encounter groundwater. In these areas, temporary control and diversion of both surface water and groundwater seepage will be necessary.

Shoring

6.122 It is estimated that the maximum depth of temporary excavation required for this project will be about 10 to 15 feet. Cantilevered or anchored steel sheet pile walls may be considered for the temporary support of excavation, depending on the required excavation depth. Cantilevered sheet pile walls are typically used for excavation depths less than 12 feet. Shoring for the UPRR culvert should be designed based on the appropriate requirements in the AREMA Manual for Railway Engineering, Chapter 8. Shoring in other areas of the alignment should be designed based on the appropriate Corps of Engineers' Engineering Manuals.

REFERENCES

AREMA 2007, American Railway Engineering and Maintenance-of-Way Association, Manual for Railway Engineering Chapter 8.

Boore, D.M., and Atkinson, G.M., 2008, Ground-motion prediction equations for the average horizontal component of PGA, PGV, and 5% damped PSA at spectral periods between 0.01s and 10.0s, Earthquake Spectra, Vol. 24, No.1, pp. 99-138

Bowles, Joseph, Foundation Analysis and Design, 5th Edition, 1997.

California Department of Conservation, Division of Mines and Geology, 2008, Guidelines for Evaluation and Mitigation of Seismic Hazards in California: Special Publication 117.

California Department of Conservation, Division of Mines and Geology, 2001, Seismic Hazard Zone Report for the Milpitas 7.5-Minute Quadrangle, Alameda and Santa Clara Counties, Seismic Hazards Zone Report 051.

California Department of Conservation, Division of Mines and Geology 1982, Revised official map of Alquist-Priolo Earthquake Fault Hazard Zones, Milpitas 7.5-Minute Quadrangle: California Division of Mines and Geology, scale 1:24,000.

Campbell, K.W., and Bozorgnia, Y., 2008, NGA ground motion model for the geometric mean horizontal component of PGA, PGV, PGD, and 5% damped linear elastic response spectra for periods ranging from 0.01s to 10.0s, Earthquake Spectra, Vol. 24, No.1, pp. 139-171.

Chiou, B.S.J., and Youngs, R.R., 2008, Chiou-Youngs NGA ground motion relations for the geometric mean horizontal component of peak and spectral ground motion parameters, Earthquake Spectra, Vol. 24, No.1, pp. 173-215.

Jennings, C.W. and Bryant, W.A., 2010, Fault Activity Map of California: California Geological Survey Geologic Data Map No. 6, scale: 1:750,000.

Lunne, R., Robertson, P.K., and Powell, J.J.M. 1997. Cone Penetration Testing in Geotechnical Practice. EF Spon/Blackie Academic, Routledge Publishers, London, 312p.

Parikh Consultants Inc. 2004. Geotechnical Office Report, Coyote and Berryessa Creek General Re-Evaluation Study for Proposed Project Modifications, Santa Clara County, California, Job No. 200145.10, April 2004.

Pradel, D., 1998a, Procedure to Evaluate Earthquake-Induced Settlements in Dry Sandy Soils: Journal of Geotechnical and Geoenvironmental Engineering, dated April, pp. 364-368.

Pradel, D., 1998b, Erratum to Procedure to Evaluate Earthquake-Induced Settlements in Dry Sandy Soils: Journal of Geotechnical and Geoenvironmental Engineering, dated October, p. 1048.

Santa Clara Valley Water District 2002. Levee Safety Technical Guidance Manual. Final Report. URS Corporation.

Southern California Earthquake Center, 1999. Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California. Dated March.

Tokimatsu, K. and H.B. Seed, 1987. Evaluation of settlements in sands due to earthquake shaking, J. Geot. Engrg., 113 (8), 861-878.

USACE, EM 1110-1-1905, "Bearing Capacity of Soils," October 30, 1992.

USACE, EM 1110-2-1902, "Slope Stability," October 3, 2003.

USACE, EM 1110-2-2100, "Stability Analysis of Concrete Structures," December 1, 2005.

USACE, EM 1110-2-2104, "Strength Design for Reinforced-Concrete Hydraulic Structures," August 20, 2003.

USACE, EM 1110-2-2502, "Retaining and Flood Walls," September 29, 1989.

USACE, EM 1110-2-1601, "Hydraulic Design of Flood Control Channels," July 1, 1991.

USACE, ER 1110-2-1150, "Engineering and Design for Civil Works Projects," August 31, 1999.

USACE, ER 1110-2-1806, "Earthquake Design and Evaluation for Civil Works Projects," July 31, 1995.

USACE, ETL 1110-2-120, "Additional Guidance for Rip Rap Channel Protection," May 14, 1971.

USACE, EM 1110-2-2902 “Engineering and Design, Conduits, Culverts and Pipes," March 31, 1998.

U.S. Geological Survey, Preliminary Geologic Map of the San Jose 30 x 60-Minute Quadrangle, California, 1999, Carl M. Wentworth, M. Clark Blake, Robert J. McLaughlin, and Russell W. Graymer, Open File Report 98-795.

Youd, T.L., and Idriss, I.M. (eds.), 1998, Summary Report in Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils: National Center for Earthquake Engineering Research Technical Report NCEER-97-0022, pp. 1-40.

Youd, T.L. and Idriss, I.M., 2001, Liquefaction Resistance of Soils: Summary report of NCEER 1996 and 1998 NCEER/SF Workshops on Evaluation of Liquefaction Resistance of Soils: Journal of Geotechnical and Geoenvironmental Engineering, dated April, pp. 297-313.

7. STRUCTURAL BASIS FOR DESIGN

GENERAL

7.1 This section presents the design criteria for the structural design of the Berryessa Creek Channel. Final analysis and design of structures have been performed to define purpose, shape, develop quantities, and to support the cost estimate.

7.2 The structural features consist of miscellaneous reinforced concrete drainage structures including floodwalls, reinforced concrete box, storm drain outlet structures, channel transition structures, and channel invert lining.

DESIGN CRITERIA

7.3 The structural design for the Berryessa Creek Channel is in accordance with the criteria set forth in EMs, ETLs, and ERs published by the Office of the Chief of Engineers and accepted engineering practice. The structural design for the UPRR box culvert at station 161+00 and the Piedmont creek culvert are in accordance with AREMA Manual of Railway Engineering.

- EC 1110-2007: Structural Design of Concrete Lined Flood Control Channels
- EM 1110-2-2103: Details of Reinforcement – Hydraulic Structures
- EM 1110-2-2104: Strength Design for Reinforced Concrete Hydraulic Structures
- EM 1110-2-2502: Retaining and Flood Walls
- EM 1110-2-2902: Conduits, Culverts, and Pipes
- ETL 1110-2-322: Retaining and Flood Walls

Loading Analysis

7.4 The current USACE requirements for design loads, load combinations, and load factors are documented in EM 1110-2-2502 Retaining and Flood Walls, EM 1110-2-2100 Stability Analysis of Concrete Structures, and EM 1110-2-2104 Strength Design Criteria for Reinforced Hydraulic Structures. The current EM 1110-2-2007 Structural Design of Concrete Lined Flood Control Channels provides additional guidance on load combinations, and along with EM 1110-2-2502 Retaining and Flood Walls, requires using at-rest earth pressures for concrete structure analyses.

7.5 EM 1110-2-2104 Strength Design Criteria for Reinforced Hydraulic Structures uses the "single load factor" method as a simplified approach to the strength design method. In addition, the basic load factor, 1.7, is multiplied by a hydraulic factor, Hf = 1.3. As such, the factored loads (U) are described by U = 1.3*(1.7*[D + L]).

Stability Requirements

7.6 The current USACE guidance in EM 1110-2-2100 Stability Analysis of Concrete Structures requires concrete structures to be designed according to loading conditions that consider sliding, flotation, and overturning/rotational modes of failure. Concrete structures must also be designed for each condition according to a factor of safety, which depends on a variety of load conditions and site-specific information.

Seismic Design

7.7 Current design criteria for the seismic evaluation are provided in ER 1110-2-1806 Earthquake Design and Evaluation for Civil Works Projects. Additional seismic design information can be found in ITL TR-05 Seismic Structural Considerations for the Stem and Base of Retaining Walls Subjected to Earthquake Ground Motions and ITL-92-11 The Seismic Design of Waterfront Retaining Structures.

CONSTRUCTION MATERIALS

Concrete

7.8 Concrete design strengths for structural elements will based on the minimum 28-day compressive strengths (f'c) as indicated below:

Miscellaneous concrete drainage structures: f'c = 4000 psi
Concrete box culvert: f'c = 4000 psi

Reinforcing Steel

7.9 Reinforcing steel will conform to ASTM A615, Grade 60, yield strength, fy = 60,000 psi.

Structural Steel.

7.10 Sheet piles shall conform to ASTM A572, Grade 50, Fy = 50 ksi.

7.11 Miscellaneous plates and shapes shall conform to ASTM A572, Grade 50, Fy = 50 ksi steel will conform to ASTM A615, Grade 60, with a yield strength, fy = 60,000 psi.

Minimum Reinforcement Cover

7.12 Per EM 1110-2-2104 Strength Design Criteria for Reinforced Hydraulic Structures, current design criteria for concrete protection of reinforcement should conform to the minimum conditions contained in Table 7.1.

Table 7.1: Minimum Concrete Clear Cover

Condition	USACE Criteria
Unformed surfaces in contact with foundation.	4 in.
Formed or screeded surfaces, such as baffle blocks and stilling basin slabs, subject to cavitation or abrasion erosion.	6 in.
Formed or screeded surfaces such as stilling-basin walls, chute-spillway slabs, and channel-lining slabs on grade.	6 in.
Equal to or greater than 24 inches in thickness.	4 in.

Condition	USACE Criteria
Greater than 12 inches and less than 24 inches in thickness.	3 in.
Equal to or less than 12-inch thickness will be in accordance with ACI 318 - #5 bar or smaller.	2.0 in. (exposed to earth) 2.5 in. (exposed to water)
Equal to or less than 12-inch thickness will be in accordance with ACI 318 - #6 bar through #18 bar.	2.0 in. (exposed to earth) 2.5 in. (exposed to water)

7.13 It is assumed that reinforced-concrete structure foundations and the channel slope paving and invert slab paving will be poured on unformed surfaces, and walls and soffits will be constructed using formwork.

RECOMMENDED SOIL DESIGN VALUES

7.14 The recommended design soil parameters are based on the test results presented in the Geotechnical Investigation Appendix of this DDR.

For miscellaneous drainage structures and box culverts:

Moist soil unit weight, γ moist = 130 pcf
Allowable Friction Coefficient = 0.30
Allowable Passive Pressure Coefficient = 1.7
Active Earth Pressure Coefficient = 0.39
At-rest Lateral Pressure Coefficient = 0.56 use in the U-frame Program and RCBC design
Earth Pressure Coefficient for Culvert = 0.33 min., 1.0 max.

STRUCTURAL ELEMENTS

Drainage Structures

7.15 Channel structures will be modeled as earth retaining or U-frame structures as shown in EM 1110-2-2007, Figure 4-1. The Corps program CUFRBC or finite element analysis will be used for the analysis and design of the U-frame structures. Concrete box culverts will be designed per AREMA Manual for Railway Engineering Chapter 8, Part 16 and Allowable Stress Design will be used in the area under the railroad tracks. The reinforced concrete design and detailing for other structures, including concrete box culverts outside of the railroad ROW shall follow the requirements of EM 1110-2-2104 that allows for the use of a single load factor of 2.21 for the final design of the concrete elements.

7.16 The transition structure beneath the Los Coches Avenue Bridge will require the removal of soil from the front of the abutment piles and could reduce the axial and lateral capacity of the abutment piles. Current seismic codes requirements will impose a much greater load on the structure than the codes that were in effect when the structure was originally designed. Cursory estimates of the structural loads on the piles indicate that the abutment foundations do not meet current standards in terms of axial resistance or lateral resistance. It is assumed at this time that the City of Milpitas understands and accepts the possibility of damage to the bridge during a seismic event. Based upon a preliminary analysis of the bridge, and the understanding that the bridge does not meet the current loading requirements, a slight reduction in axial capacity will not significantly affect the conclusions and/or performance of the bridge.

7.17 With the assumption that the bridge will act like a concrete diaphragm and will transfer the longitudinal seismic load into the soil behind the abutment. The at-rest resistance will be engaged at the onset of any movement. The onset of movement in this case will be defined as equivalent to 0.1% of the abutment height. Because of the small amount of movement required to engage the at-rest resistance, we will assume to full depth of the abutment will push against the soil backfill. With an abutment height of 8.6 feet, and the calculated movement based on 0.1% of the height, a value of 0.1 inches has been estimated as the movement needed to engage the at-rest resistance. The at-rest resisting value has been calculated as 4,440 pounds per foot of abutment width. The as-built plans show the bridge abutment width at 48 feet, which equates to a total resistance of 213 kips. This exceeds the estimated imposed seismic load of 199 kips.

7.18 Since the abutment needs to move approximately 0.1 inches to engage the at-rest pressure, the piles would also need to move this distance. At this amount of deflection, the existing piles would see an approximate maximum load of 6.6 kips of shear and a moment of 9.92 kip-ft. These values were provided and calculated by an L-Pile program. Based the analysis, the existing piles should each be adequate for approximately 8 kips of shear and 22 kip-ft of moment, which both exceed the calculated values for a deflection of 0.1 inches. Assuming the existing piles obtain their vertical capacity through skin friction, the pile vertical capacity would probably be reduced by about 6% due to the removal of some soil in front of the piles. We do not believe that this amount of reduction in the vertical pile capacity is significant enough to compromise the vertical load carrying capacity of the piles.

7.19 The reduced axial and lateral resistance of the bridge foundations may lead to lateral deflections during a seismic event. The magnitude of this deflection cannot be accurately determined without a very detailed structural study of the bridge. However, the abutment deflection could impact the transition structure and possibly damage or crack the transition structure. The abutment piles' reduced axial and lateral capacity may also be an issue for the existing bridge during a seismic event. The reduced capacity caused by transition structure construction may result in possible damage to the bridge or its piles during a seismic event. Therefore, the transition structure beneath Los Coches Bridge will be designed to accommodate some movement from the bridge abutment piles. A letter regarding the additional analysis was prepared and submitted to the City of Milpitas for their review as part of the permitting process through the City. This letter and analysis is included in the structural appendix of this DDR.

REFERENCE DOCUMENTS

EM 1110-2-2000, Standard Practice for Concrete for Civil Works Structures, U.S. Army Corps of Engineers, 1 Feb. 94.

EM 1110-2-2007, Structural Design of Concrete Lined Flood Control Channels, U.S. Army Corps of Engineers, 30 Apr. 95.

EM 1110-2-2100, Stability Analysis of Concrete Structures, U.S. Army Corps of Engineers, 1 Dec. 05.

EM 1110-2-2104, Strength Design for Reinforced – Concrete Hydraulic Structures, U.S. Army Corps of Engineers, 20 Aug 03.

EM 1110-2-2502, Retaining and Flood Walls, U.S. Army Corps of Engineers, 29 Sep. 89.

ETL 1110-2-322, Retaining and Flood Walls, U.S. Army Corps of Engineers, 15 Oct. 90.

ETL 1110-2-340, Structured Design of Reinforced Concrete U-Shaped Channels, U.S. Army Corps of Engineers, 31 Mar. 93.

ER 1110-2-1150, Engineering and Design for Civil Works Projects, U.S. Army Corps of Engineers, 31 Mar. 94.

Building Code Requirements for Reinforced Concrete (ACI 318-11) and Commentary (ACI 318R-11), American Concrete Institute, 2014.

AREMA 2012, Manual for Railway Engineering, American Railway Engineering and Maintenance-of-Way Association, 2012

AASHTO LRFD Bridge Design Specifications, Sixth Edition, 2012 American Association of State Highway and Transportation Officials, with California Amendments (AASHTO-CA BDS-6).

LRFD Guide Specifications for the Design of Pedestrian Bridges, 2009 American Association of State Highway and Transportation Officials.

Standard Specifications 2010 State of California, Department of Transportation.

Standard Plans 2010 State of California, Department of Transportation.

Building Code Requirements for Structural Concrete, ACI 318-11 American Concrete Institute.

2013 California Building Code California Building Standards Commission.

Minimum Design Loads for Buildings and Other Structures (ASCE 7-10) American Society of Civil Engineers.

8. CARE OF TRAFFIC DURING CONSTRUCTION

8.1 A haul route and traffic plan is included in the 90% design plans. This plan was developed with close coordination with the City of Milpitas and is used to guide the construction contractor in his development of a traffic control plan. Prior to start of construction, the construction contractor will develop and submit a signed and stamped temporary traffic control plan for approval by the City of Milpitas.

8.2 In general, haul routes will utilize Milpitas Boulevard as a primary connector between the project site and freeway access (via I-680) along East Calaveras and Montague Expressway. Channel access points will occur at Los Coches, Yosemite Drive, Ames Ave, Milpitas Boulevard, and Pecten Court. Additionally, access at East Calaveras will be exit only. Flagmen will be required while construction traffic is active at these location.

8.3 For Los Coches Street, no lane closures will be allowed and street parking will be limited between the hours of 7:00 am to 6:00 pm.

8.4 For Yosemite Drive and Ames Avenue, shoulder and lane closures will be allowed, but traffic closures must maintain one open lane in each direction during construction. Overnight parking closures are allowed, but no equipment or material storage will be permitted on City of Milpitas streets.

8.5 During partial lane closures, it would be necessary to close the sidewalk on one side of the street at each location for the safety of pedestrians; pedestrians would need to detour to the sidewalk on the other side of the street.

9. CARE AND DIVERSION OF WATER DURING CONSTRUCTION

9.1 All dewatering activities would be temporary in nature, confined to a small area, and occur mostly during dry season months (mid-April to mid-October). Accumulated water would be diverted around the work area. The creek flow would be temporarily diverted around the work area by using one of the following types of diversions: temporary durable plastic K-rail barrier system, water-tight cofferdam, or inflatable bladder dam. These diversions would remain in place throughout the in stream construction period. The locations and spacing of the diversions would be determined based on the type and length of construction activity. Best management practices (BMPs) would be implemented to reduce impacts on groundwater supplies and discharge

9.2 In order to limit dewatering operations construction of the project where groundwater is expected to be encountered should be constructed early in the project when groundwater levels are lowest.

9.3 For excavation that takes place above the Jones Chemical plume, as mentioned in Section 4 of the DDR, the Area of Interest (AOI) is approximately 25 feet wide by 900 feet long, or approximately from Station 156+00 to Station 165+00. This AOI is where channel excavation takes place above the plume.

9.4 If groundwater is exposed in the AOI during excavation, it shall be assumed that the groundwater is contaminated and must be specifically managed per the attached Groundwater Management Plan (in **Appendix E**), which includes requirements for diversion and control of groundwater; groundwater treatment, storage, and conveyance; compliance monitoring; reporting; and demobilization of the treatment facilities.

10.CARE OF HABITAT DURING CONSTRUCTION

10.1 The existing habitat in the project area consists of a sparse cover of herbaceous vegetation and nonnative grasses. Herbaceous vegetation would be removed during construction; however, the project reach would be revegetated by hydroseeding after construction. Corps' guidance would require removing woody vegetation on the levee prism and within 15 feet of the toe of the levee/floodwall. At this time, the project does not include a levee section so removal of trees for this purpose is not expected.

10.2 A current tree survey was performed by SCVWD to identify any native trees that will require removal for construction of the channel. Native trees that are impacted and require removal will be identified and replanting will be provided based on the project EIS/EIR and additional coordination with the Fish and Wildlife service. A certified arborist will identify and mark the trees to be protected in place prior to construction. Additionally, clearances for the riparian habitat upstream of station 191+77 will be provided to limit construction work in this area.

10.3 Some trees will require biodegradable erosion protection and minor grading around the tree and root system in order to provide erosion protection. The proposed protection is planned to limit excavation around the tree to reduce stress impacts to the tree. Native trees where protection is provided will be monitored for a year to confirm that the construction and channel improvements have not led to the determent of the native tree health. If the construction is determined to lead to the detrimental impacts, the native tree will be removed and replaced based upon the project EIS/EIR.

10.4 Trees will be replaced based on coordination with the Fish and Wildlife Service. The replacement rate for native trees removed with a diameter at breast height (dbh) of 2 inches up to 8 inches will be to plant 1 native tree; native trees removed with a diameter up to 20 inches dbh will be replaced with 2 native trees; native trees greater than 20 inches dbh will be replaced with 3 native trees; and native shrubs removed will be replaced with 2 native shrubs. Replacement of the affected trees and shrubs per the above ratio results in replanting 46 native shrubs and 60 native trees. In addition, the project will overplant an additional 55% of trees and shrubs to account for expected mortality resulting in a total of 81 shrubs and 83 trees. Native trees and shrubs will be harvested via seed or cuttings from the Berryessa Creek Corridor and the Coyote Creek watershed to protect the genetic characteristics of the area. Seeds or cuttings from some species will be grown into container stock, while acorns and buckeye seed will be directly planted to limit tap root damage, pathogens, and enhance long-term survival. Additionally, any removal and/or trimming of trees and shrubs will be supervised and/or completed by a certified arborist.

10.5 If a listed species is encountered during excavations or any project activities, activities would cease until the species is removed and relocated by a USFWS-approved biologist. Any incidental take would be reported to the USFWS immediately by telephone.

10.6 The work area footprint will be limited to avoid unnecessary destruction of native plants and species. All required tree removal activities will be performed by or under the direct supervision of a certified arborist. The specific area will be identified in the plans and specifications and within the *Engineering Considerations and Instructions for Field Personnel.*

11.DISPOSAL OF MATERIALS

11.1 All materials that need to be disposed of off-site are assumed to be loaded and hauled to the Newby Island Sanitary Landfill located in Milpitas, California except as indicated in paragraph 11.2 and 11.3, below. This landfill, located approximately five miles from the project site, would be able to take in the excess excavated material and any demolished material that would be removed during construction.

11.2 Ten percent (10%) of material is assumed Class II and will be go to the Altamont landfill in Livermore, California. This landfill is approximately 40 miles from the project site.

11.3 Based upon HTRW Soil Sampling Report, Upper Berryessa Creek Flood Risk Management Project between Montague Expressway and Yosemite Drive, Santa Clara County, Milpitas, California" (prepared by Tetra Tech in January 2015) contaminated soil is not expected to be encountered. If during construction contaminated soils are encountered, the soil will be removed and stockpiled on the JCI Jones Site for disposal by others.

11.4 For excavation that takes place above the Jones Chemical plume, as mentioned in Section 4 of the DDR, the Area of Interest (AOI) is approximately 25 feet wide by 900 feet long, or approximately from Station 156+00 to Station 165+00. This AOI is where channel excavation takes place above the plume.

11.5 If groundwater is exposed in the AOI during excavation, it shall be assumed that the groundwater is contaminated and must be specifically managed per the attached Groundwater Management Plan (in **Appendix E**), which includes requirements for diversion and control of groundwater; groundwater treatment, storage, and conveyance; compliance monitoring; reporting; and demobilization of the treatment facilities.

12. ENVIRONMENTAL

BIOLOGICAL RESOURCES PER GRR

12.1 The preliminary design included in the GRR identified vegetation along the channel to be removed prior to construction. This included approximately 15 trees, located between Calaveras Boulevard and Los Coches Street, requiring removal for construction access. These trees are located on the landside of the proposed floodwall. Landside trees include occasional small patches of nonnative and/or invasive trees including Eucalyptus, Black Acacia, Mexican palm, Australian willows, fruit trees, and ornamental trees. The removal of landside vegetation and woody vegetation in the project area would not substantially interfere with the movement of resident or migratory birds.

12.2 In a 2005 wetland delineation performed by the Corps found that up to 0.39 acre of wetland may occur in the proposed project area. A subsequent wetland delineation update in 2014 found that no wetlands fell under the jurisdiction of the Corps, and less than 0.5 acres of non-jurisdictional fringing wetland occur in the stretch of stream between Ames Avenue and Calaveras Boulevard and within the lower 80 feet of Piedmont Creek. Areas of these streams at or below the Ordinary High Water Mark qualify as waters of the U.S. and are found throughout the project area, including the lower 400 feet of Los Coches Creek and the lower 80 feet of Piedmont Creek. Waters of the U.S. within the project area amount to approximately 4.05 acres.

12.3 Since the preparation of the GRR, the project team has performed additional coordination with Fish and Wildlife service to revise the tree replanting requirements and plan. This included the preparation of a Tree and Shrub Survey and a native Trees/shrubs replanting memorandum (Appendix E-1). The results are summarized below in the Riparian Habitat section below.

WETLAND AREAS

12.4 The 2014 Wetland Delineation Report (Tetra Tech 2014) identified 0.0197 acre of riverine herb dominated wetland occurring adjacent to and downstream of the bridge at Calaveras Boulevard. These wetlands are outside of the project area and will not be affected during construction. The report also identified less than 0.5 acre of non-jurisdictional fringing wetland in the Upper Berryessa Creek channel between Ames Avenue and Calaveras Boulevard and within the lower 80 feet of Piedmont Creek. A high level of function will be accomplished by widening the existing channel, increasing the overall area that is expected to have hydrology supportive of wetland vegetation. The proposed improvements include the hydro seeding of all channel banks with a native grass mix above the three (3) foot level, with the lower three (3) feet of bank hydro seeded with native wetland species. Based upon these features, no further mitigation is planned for these fridge wetlands since they will return after construction is completed.

WATER QUALITY

12.5 A Groundwater Management Plan (GWMP) has been prepared and included as an appendix (E.3) to these Specifications to guide field activities within the Jones Chemical, Inc.

(JCI) groundwater plume during implementation of the Project. The GWMP shall be implemented by the Construction Contractor. In general, the GWMP defines the approach for the extraction, conveyance, and treatment of groundwater within the Area of Interest (AOI) that is bound by the intersection of the Project and the JCI groundwater plume, as shown on the plans. This GWMP does not apply to any work performed outside the AOI and will only be utilized if groundwater is encountered by the Construction Contractor while performing work within the AOI.

12.6 Upon coordination and processing of the 60% plans through the San Francisco Bay Regional Water Quality Control Board (Water Board), it was apparent that they were opposed to the use of cellular bank material as part of the project's erosion protection measures. The Water Board requested more natural protection measures. Due to the high velocities and shear stresses within the channel, simply removing the cellular bank protection is not an acceptable solution. In consultation with the Project Delivery Team (PDT), the design was revised to having three sections of erosion protection: First would be the original buried rip rap revetment that primarily provides toe protection. This rip rap toe revetment was adjusted to a lower elevation based upon the shear stresses near or about 2 lbs/ft^2. Second is a section of buried rip rap revetment—with smaller rock than the first section—in lieu of the cellular bank protection. The third section consists of natural native grasses that are able to withstand the shear stresses at the upper portions of the banks without any supplemental buried revetment. Temporary erosion protection will be provided during the first approximately three years after construction through the use of a bio-degradable erosion protection that will help to increase the erosion resistance during the establishment period of hydro-seeded native grasses. The use of the buried rip rap will stabilize the bank and prevent the bank caving which has contributed to the persistent erosion issues along this reach of the channel.

12.7 Additional comments from the Water Board consist of a request to replace the Los Coches Creek and Piedmont Creek RCB culverts with free spanning bridges or earthen bottom culverts. These options were discussed with the PDT and determined to be impractical due to the sewer lines, high velocities and shear stresses in this area. The existing channels are vulnerable to scour and erosion that would expose an existing sewer line at Los Coches and erode the left bank at Piedmont Creek. The use of a clear span would not provide the invert stabilization needed to prevent further erosion in these areas. The 15" sewer line crossing at each location causes a 2 foot drop into the main channel and would not allow for a deeper culvert to be used. This drop also greatly increases turbulence, channel velocities, and erosion potential. Therefore, the culvert design protects the existing sewer lines and modifies the confluence angle from 90 degrees to 30 degrees so that the hydraulic design conforms to USACE requirements. The use of the culverts and revised confluence angles reduces erosion issues and results in a lower water surface (about 1.5 feet), thus limiting the need for additional floodwalls and other structural/concrete improvements.

RIPARIAN HABITAT

12.8 Approximately 0.18 acre of riparian habitat is found below the top of bank on the left bank of Upper Berryessa Creek in Reach 4 (upstream of station 191+77). This area will be avoided during construction, and a buffer zone will be established around the riparian area to avoid damage to the roots of the trees. These measures will ensure that impacts to riparian habitat are minimized.

12.9 Some native trees will require minor grading (depths of 9”-16”) around the tree and root system for erosion protection and a small number of additional trees will require removal. A tree survey prepared by the SCVWD (August 2015) identified all existing trees and shrubs within the project area and found that 53 native trees and shrubs would be affected by the construction. Replanting or protecting-in-place for impacts to native trees and shrubs will be undertaken as described below.

12.10 Trees will be replanted based on coordination with the Fish and Wildlife Service. The replacement rate for native trees removed with a diameter at breast height (dbh) of 2 inches up to 8 inches will be to plant 1 native tree; native trees removed with a diameter up to 20 inches dbh will be replaced with 2 native trees; native trees greater than 20 inches dbh will be replaced with 3 native trees; and native shrubs removed will be replaced with 2 native shrubs. Replacement of the affected trees and shrubs per the above ratio results in replanting 46 native shrubs and 60 native trees. In addition, the project will overplant an additional 55% of trees and shrubs to account for expected mortality resulting in a total of 81 shrubs and 83 trees. Native trees and shrubs will be harvested via seed or cuttings from the Berryessa Creek Corridor and the Coyote Creek watershed to protect the genetic characteristics of the area. Seeds or cuttings from some species will be grown into container stock, while acorns and buckeye seed will be directly planted to limit tap root damage, pathogens, and enhance long-term survival. Additionally, any removal and/or trimming of trees and shrubs will be supervised and/or completed by a certified arborist.

12.11 Dewatering is not expected to be needed in the riparian habitat vicinity. If required, the dewatering will occur as late in the construction season as possible to minimize the duration of stress on the trees before the rainy season starts.

AIR QUALITY ANALYSIS

12.12 USACE air quality modeling and the USACE’s Final EIS (USACE 2014) indicate that the ongoing design of the Upper Berryessa Creek project should allow construction to proceed while maintaining project emissions below thresholds of significance. Therefore, BAAQMD guidelines indicate this project and similar below thresholds of significance projects must employ the following basic construction mitigation measures:

- All exposed surfaces (e.g., parking areas, staging areas, soil piles, graded areas, and unpaved access roads) shall be watered two times per day.
- All haul trucks transporting soil, sand, or other loose material off-site shall be covered.
- All visible mud or dirt track-out onto adjacent public roads shall be removed using wet power vacuum street sweepers at least once per day. The use of dry power sweeping is prohibited.
- All vehicle speeds on unpaved roads shall be limited to 15 mph.

- All roadways, driveways, and sidewalks to be paved shall be completed as soon as possible. Building pads shall be laid as soon as possible after grading unless seeding or soil binders are used.

- Idling times shall be minimized either by shutting equipment off when not in use or reducing the maximum idling time to 5 minutes (as required by the California airborne toxics control measure Title 13, Section 2485 of California Code of Regulations [CCR]). Clear signage shall be provided for construction workers at all access points.

- All construction equipment shall be maintained and properly tuned in accordance with manufacturer‘s specifications. All equipment shall be checked by a certified visible emissions evaluator.

- A publicly visible sign shall be posted that includes the telephone number and person to contact at the lead agency regarding dust complaints. This person shall respond and take corrective action within 48 hours. The Air District‘s phone number shall also be visible to ensure compliance with applicable regulations.

12.13 In addition to these basic measures, the construction contractor should also avoid use of portable generators where less polluting energy sources are available.

12.14 If final project design indicates conditions adversely affecting validity of existing AQ modeling, a reanalysis may be needed. Also, the contractor would be required, pre-construction, to inform BAAQMD on total emitting equipment to be operated and the construction area to be disturbed per day in order to verify modeling validity.

12.15 If it should be determined on re-analysis that emissions may exceed threshold of significance, many additional and more stringent mitigations are applicable, and would be added to the above basic measures.

13.REAL ESTATE PLAN

13.1 The Real Estate Plan was prepared by the Corps (2013) for inclusion with the GRR-EIS. Some properties required for and/or impacted by the project are located within, adjacent to or close to SCVWD's existing ROWs along Berryessa Creek, primarily downstream of I-680. Currently, SCVWD is the fee owner of 15.88 acres of the required 25 acres for permanent project acquisition needs. The remaining 9.12 acres required for (permanent) channel improvement easement (CIE) are owned as identified above. Twenty-five acres of land will be required for CIEs, 11.91 acres will be required for temporary work area easement (TWAE), and 2.08 acres for flood protection levee easement (FPLE) for the required floodwalls. Of the nine parcels that will be encumbered by TWAEs, four parcels will be required for staging areas consisting of 7.6 acres, and five parcels will be required to support construction consisting of 4.31 acres.

13.2 The total cost estimate for real estate requirements for the project is estimated at $10,219,000 (includes a 15.0 percent contingency) and includes the value of sponsor-owned estates required for the project.

13.3 The project would also require relocations of publicly and privately owned utilities. The relocation is estimated at $2,220,000 (includes a 23.82 percent contingency).

14. COST ESTIMATES

FIRST COSTS

14.1 The estimated first costs shown in Table 14.1 were developed using Fiscal Year 2015 price levels. They include estimates for construction for flood risk management; relocations; plantings; planning, engineering, and design; and construction management.

14.2 Unit costs were developed using MCACES 2nd Generation (MII) estimating software in accordance with guidance contained in ER 1110-2-1302, Civil Works Cost Engineering. Along with vendor quotes, the software utilized the MCACES 2012 English Unit Cost Library, 2016 Local Bay Area Labor Library, and the 2014 Equipment Library (Region VII) for the base cost estimates. Contingency allowances were assigned for each item of the cost estimate. Costs for lands and damages were provided by the U.S. Army Corps of Engineers – San Francisco District. Costs for planning, engineering, and design, and construction management are based on estimated costs to complete the project.

Table 14.1 Design Documentation Report Cost Estimate (1 Oct 2015 Effective Price Level Date)

Account No.	Description	Amount ($1,000)
01	Lands and Damages	$10,219
02	Relocations	$2,220
06	Fish & Wildlife Facilities	$379
08	Roads, Railroads and Bridges	$755
11	Levees and Floodwalls	$16,706
18	Cultural Resources	$122
30	Planning, Engineering, and Design	$4,864
31	Construction Management	$1,832
	PROJECT COST TOTALS	**$37,097**

ANNUAL OPERATION, MAINTENANCE, REPAIR, REPLACEMENT, AND REHABILITATION (OMRR&R) COSTS

14.3 Operation and maintenance (O&M) activities would occur after the project is constructed in order to keep project features functioning as designed. Annual inspections of vegetation, bridges, culverts, and channel reaches will be conducted. Vegetation control, partial vegetation replacement, trash and debris removal, and periodic structural maintenance will be required. Other activities will include maintenance and repair of the channel bank protection, graffiti removal, encroachment removal to preserve clear zones and channel, and access road maintenance.

14.4 Key maintenance tasks will be sediment removal from the channel and scour hole repairs. Since 1977, an annual average of approximately 7,000 cubic yards of sediment and debris has been removed from Berryessa Creek upstream of Calaveras Boulevard. It estimated that sediment removal would occur at an interval as often as (To be provided with O&M plan). However, when several low-flow years occur sequentially, sediment removal may occur every (To be provided with O&M plan).

14.5 An O&M manual will be prepared prior to the flood risk management improvements construction in accordance with ER 1130-2-304, Project Operations, and the applicable provisions of ER 1150-2-301, Local Cooperation. The non-federal sponsor, SCVWD, will be responsible for the operation and maintenance of the flood risk management improvements.

15. PLAN RESPONSIBILITIES

COST APPORTIONMENT

15.1 The costs for the Berryessa Creek project were allocated to a single purpose of flood risk management. The Certified Project Costs are presented in **Error! Reference source not found.**.

Table 15-1 Certified Project Costs (1 Oct 2015 Effective Price Level Date)		
WBS No.	**Description**	**Amount ($1,000)**
01	Lands and Damages	$10,219
02	Relocations	$2,220
06	Fish & Wildlife Facilities	$379
08	Roads, Railroads and Bridges	$755
11	Levees and Floodwalls	$16,706
18	Cultural Resources	$122
30	Planning, Engineering, and Design	$4,864
31	Construction Management	$1,832
	PROJECT COST TOTALS	**$37,097**

FEDERAL RESPONSIBILITIES

15.1 The federal government will be responsible for:

- Providing detailed design, construction, and construction administration necessary for implementation of a flood risk management project.
- Evaluating and determining applicability of credit against local sponsor's costs for the project pursuant to Section 104 of PL 99-662. The amount of credit, where applicable, will be dependent on the actual construction cost of the project and verification of costs for those items submitted by local sponsors for credit.
- Developing a management plan for operation and maintenance.

NON-FEDERAL RESPONSIBILITIES

15.2 The non-federal interests will be responsible for:

- Providing, during construction, a cash contribution equal to 5 percent of total project costs.
- Providing all lands, easements, and ROW, including suitable borrow and dredged or excavated material disposal areas, and perform or assure the performance of all relocations,

except railroads, determined by the Government to be necessary for the construction, operation, and maintenance of the project.

- Providing or paying the government the cost of providing all retaining dikes, waste weirs, bulkheads, and embankments, including all monitoring features and stilling basins, that may be required at any dredged or excavated material disposal areas required for the construction, operation, and maintenance of the project.

- Allowing the government a right to enter, at reasonable times and in a reasonable manner, upon land that the local partner owns or controls for access to the project for the purpose of inspection, and, if necessary, for the purpose of completing, operating, maintaining, repairing, replacing, or rehabilitating the project.

- Assuming responsibility for operating, maintaining, replacing, repairing, and rehabilitating the project or completed functional portions of the project, including mitigation features, without cost to the government, in a manner compatible with the project's authorized purpose and in accordance with applicable Federal and State laws and specific directions prescribed by the government in the OMRR&R manual and any subsequent amendments thereto.

- Holding and sparing the government from all damages arising from the construction, operation, maintenance, repair, replacement, and rehabilitation of the project and any project-related betterments, except for damages due to the fault or negligence of the government or the government's contractors.

- Performing, or causing to be performed, any investigations for hazardous substances that are determined necessary to identify the existence and extent of any hazardous substances regulated under the Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA), 42 USC 9601-9675, that may exist in, on, or under lands, easements, or ROW necessary for the construction, operation, and maintenance of the project, except that the non-federal partner shall not perform such investigations on lands, easements, or ROW that the government determines to be subject to the navigation servitude without prior specific written direction by the government.

- Assuming complete financial responsibility for all necessary cleanup and response costs for any CERCLA-regulated materials located in, on, or under lands, easements, or ROW that the government determines necessary for the construction, operation, or maintenance of the project.

- Providing the non-Federal share of that portion of the costs of archeological data recovery activities associated with historic preservation, that are in excess of 1 percent of the total amount authorized to be appropriated for the project, in accordance with the cost sharing provisions of the agreement.

16. IMPLEMENTING THE PLAN

PREPARATION OF PLANS AND SPECIFICATIONS

16.1 The plans and specifications was completed in April 2016.

SCHEDULE OF DESIGN AND CONSTRUCTION

16.2 The project is scheduled to be advertised for construction in Fiscal Year 2016 and awarded in Fiscal Year 2016. It is estimated that overall construction would take up to 18 months to construct and is estimated to be completed in Fiscal Year 2018.

17. PROJECT PARTNERSHIP AGREEMENT

17.1 The SCVWD has provided a letter of intent acknowledging sponsorship requirements for the Berryessa Creek Project. Prior to the start of construction, the non-Federal sponsor will be required to enter into an agreement with the Federal Government to comply with Section 221 of the Flood Control Act of 1970 (PL 91-611), and the Water Resources Development Act of 1986 (PL 99-662).

18. RECOMMENDATIONS

18.1 This DDR describes in detail the general design, including departures from the previously approved plan, of the portion of the Berryessa Creek Flood Risk Management Project. It is recommended that this report provide the basis for the development of plans and specifications for the construction of the Berryessa Creek Flood Risk Management Project.

18.2 The estimated total cost of the project is currently at $37,097,000 under 2015 prices.